ΕΛΛΗΝΙΚΗ
ΕΠΙΣΤΗΜΟΝΙΚΗ
ΕΤΑΙΡΕΙΑ
ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ
& ΓΕΩΤΕΧΝΙΚΗΣ
ΜΗΧΑΝΙΚΗΣ

Wave Rock - Western Australia

Τα Νέα της Ε Ε Ε Ε Γ Μ

42

Αρ. 42 – ΔΕΚΕΜΒΡΙΟΣ 2011

SIMSG ISSMGE

Η Εκτελεστική Επιτροπή της ΕΕΕΕΓΜ
εύχεται σε όλους Καλή Χρονιά

Π Ε Ρ Ι Ε Χ Ο Μ Ε Ν Α

Ε Π Ε Ι Γ Ο Ν

18th International Conference on
Soil Mechanics and Geo-technical Engineering
"Challenges and Innovations in Geo-technics"
1 – 5 September 2013, Paris, France

Πρόσκληση υποβολής περιλήψεων άρθρων (σελ. 35).

ΠΡΟΓΡΑΜΜΑ ΕΚΔΗΛΩΣΕΩΝ ΠΕΡΙΟΔΟΥ ΙΑΝΟΥΑΡΙΟΥ – ΙΟΥΝΙΟΥ 2012

Η προετοιμασία και η διεξαγωγή του 6ου Πανελληνίου Συνεδρίου Γεωτεχνικής και Γεωπεριβαλλοντικής Μηχανικής τον Σεπτέμβριο – Οκτώβριο 2010 και του XV European Conference on Soil Mechanics and Geotechnical Engineering τον Σεπτέμβριο 2011 περιώρισαν τις άλλες εκδηλώσεις τηε ΕΕΕΕΓΜ τα δύο προηγούμενα χρόνια. Με το νέο έτος, απαλλαγμένοι από τις παραπάνω υποχρεώσεις, προχωράμε σε μια σειρά εκδηλώσεων, που πιστεύουμε ότι τα έλξουν το ενδιαφέρον των μελών και των φίλων της ΕΕΕΕΓΜ. Το πρόγραμμα της περιόδου Ιανουαρίου – Ιουνίου 2012, στο οποίο παραθέτουμε και τις εκδηλώσεις άλλων φορέων που ενδεχομένως ενδιαφέρουν τα μέλη μας, έχει ως εξής:

ΙΑΝΟΥΑΡΙΟΣ

Τετάρτη 11
Διοργάνωση: ΤΟΜΕΑΣ ΓΕΩΤΕΧΝΙΚΗΣ ΕΜΠ
«**Καταστατικοί Νόμοι, Αναγκαιότητα και Χρησιμότητα**»
ΚΟΛΥΜΠΑΣ Δημήτριος, Δρ. Πολιτικός Μηχανικός, Professor, Geotechnical and Tunnel Engineering, University of Innsbruck

Πέμπτη 12
Διοργάνωση: ΤΟΜΕΑΣ ΓΕΩΤΕΧΝΙΚΗΣ ΕΜΠ
«**Μη Γραμμική Αλληλεπίδραση Εδάφους - Κατασκευής υπό Ισχυρή Σεισμική Ροπή**» - Παρουσίαση Διδακτορικής Διατριβής
ΑΠΟΣΤΟΛΟΥ Μάριος, Πολιτικός Μηχανικός

ΦΕΒΡΟΥΑΡΙΟΣ

Δευτέρα 06 ΑΘΗΝΑ
Διοργάνωση: ΕΕΕΕΓΜ
«**The flexible structural facing for the rockfall protection and slope stability – Design approach and new calculation concepts**»
MACCAFERRI

Τρίτη 07 ΘΕΣΣΑΛΟΝΙΚΗ
Διοργάνωση: ΕΕΕΕΓΜ
«**The flexible structural facing for the rockfall protection and slope stability – Design approach and new calculation concepts**»
MACCAFERRI

Τετάρτη 22
Διοργάνωση: ΕΕΕΕΓΜ
ΑΘΗΝΑΪΚΗ ΔΙΑΛΕΞΗ ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ
«**Διαχείριση της αβεβαιότητας στη Γεωτεχνική Μηχανική – Ο ρόλος της Ενόργανης Παρακολούθησης και των Μετρήσεων**»
ΤΣΟΤΣΟΣ Στέφανος, Δρ. Πολιτικός Μηχανικός, Καθηγητής Τομέα Γεωτεχνικής Μηχανικής Τμήματος Πολιτικών Μηχανικών Πολυτεχνικής Σχολής Αριστοτελείου Πανεπιστημίου Θεσσαλονίκης

Τετάρτη 29 ΑΘΗΝΑ
Διοργάνωση: ΕΕΕΕΓΜ
«**Reinforced Pavements**»
BENNETT Richard – Δρ. Πολιτικός Μηχανικός - Μεταλλειολόγος, Group Pavement Engineer, Maccaferri UK Ltd

ΜΑΡΤΙΟΣ

Πέμπτη 01 ΘΕΣΣΑΛΟΝΙΚΗ
Διοργάνωση: ΕΕΕΕΓΜ
«**Reinforced Pavements**»
BENNETT Richard – Δρ. Πολιτικός Μηχανικός - Μεταλλειολόγος, Group Pavement Engineer, Maccaferri UK Ltd

Δευτέρα 12
Διοργάνωση: ΕΕΕΕΓΜ
«**Η Ευστάθεια Φυσικών Πρανών και Ορυγμάτων σε Στιφρές Αργίλους**»
ΜΠΕΛΟΚΑΣ Γεώργιος – Δρ. Πολιτικός Μηχανικός, ΚΕΠΕ ΔΕΗ

Πέμπτη 22 ÷ Παρασκευή 23
Διοργάνωση: ΕΛΛΗΝΙΚΗ ΕΠΙΤΡΟΠΗ ΣΗΡΑΓΓΩΝ ΚΑΙ ΥΠΟΓΕΙΩΝ ΕΡΓΩΝ (ΕΕΣΥΕ)
International Symposium "Practices and trends for financing tunnels and underground works"

Τετάρτη 26 ΑΘΗΝΑ
Διοργάνωση: ΕΕΕΕΓΜ
SALZMANN Hannes, GEOBRUGG

Πέμπτη 27 ΘΕΣΣΑΛΟΝΙΚΗ
Διοργάνωση: ΕΕΕΕΓΜ
SALZMANN Hannes, GEOBRUGG

ΑΠΡΙΛΙΟΣ

Δευτέρα 23
Διοργάνωση: ΕΕΕΕΓΜ
«**Η παραμένουσα αντοχή συνεκτικών εδαφών**»
ΤΙΚΑ Θεοδώρα – Δρ. Πολιτικός Μηχανικός, Καθηγήτρια Τομέα Γεωτεχνικής Μηχανικής Τμήματος Πολιτικών Μηχανικών Πολυτεχνικής Σχολής Αριστοτελείου Πανεπιστημίου Θεσσαλονίκης

ΜΑΙΟΣ

Τρίτη 8
ΓΕΝΙΚΗ ΣΥΝΕΛΕΥΣΗ – ΑΡΧΑΙΡΕΣΙΕΣ

Τετάρτη 23
Διοργάνωση: ΕΕΕΕΓΜ
«**Αντιμετώπιση ολίσθησης σε αυτοκινητόδρομο υπό κυκλοφορία: επίχωμα Ε6 Εγνατίας οδού στην περιοχή Γρεβενών**»
ΣΑΚΟΥΜΠΕΝΤΑ Ελένη – Πολιτικός Μηχανικός, M.Sc, Τμήμα Μελετών της ΕΓΝΑΤΙΑ ΟΔΟΣ Α.Ε.

ΙΟΥΝΙΟΣ

Δευτέρα 11
Διοργάνωση: ΕΕΕΕΓΜ
«**Δυσχέρειες στην αντιμετώπιση σύνθετων προβλημάτων της γεωτεχνικής σεισμικής μηχανικής με τον EC8**»
ΨΑΡΡΟΠΟΥΛΟΣ Πρόδρομος – Δρ. Πολιτικός Μηχανικός, Αναπληρωτής Καθηγητής στο Τμήμα Μηχανικών Αεροπορικών Εγκαταστάσεων (Πολιτικών Μηχανικών) Σχολής Ικάρων

ΠΕΡΙΛΗΨΕΙΣ ΠΡΟΣΦΑΤΩΝ ΔΙΔΑΚΤΟΡΙΚΩΝ ΔΙΑΤΡΙΒΩΝ ΕΛΛΗΝΙΚΩΝ ΠΟΛΥΤΕΧΝΙΚΩΝ ΣΧΟΛΩΝ

ΣΥΣΤΗΜΑΤΑ ΟΛΙΣΘΗΣΕΩΣ ΥΠΟΒΑΛΛΟΜΕΝΑ ΣΕ ΕΓΓΥΣ-ΤΟΥ-ΡΗΓΜΑΤΟΣ ΣΕΙΣΜΙΚΕΣ ΔΙΕΓΕΡΣΕΙΣ: ΑΝΑΠΤΥΞΗ ΘΕΜΕΛΙΩΔΩΝ ΑΝΕΛΑΣΤΙΚΩΝ ΑΝΑΛΟΓΩΝ

Ευαγγελία Γαρίνη
Εθνικό Μετσόβιο Πολυτεχνείο
Σχολή Πολιτικών Μηχανικών

Επιβλέπων: **Καθηγητής Γεώργιος Γκαζέτας**

Ο θεματικός πυρήνας της διατριβής είναι η διερεύνηση της σεισμικής απόκρισης θεμελιωδών εντόνως ανελαστικών συστημάτων όταν αυτά διεγείρονται όχι από οποιαδήποτε επιταχυνσιογραφήματα αλλά από καταγραφές εγγύς-του-ρήγματος. Τα εν λόγω επιταχυνσιογραφήματα εμφανίζουν παλμούς λόγω *"κατευθυντικότητας"* ή και *"αλτηματικότητας"*, σαφείς ενδείξεις της κατεύθυνσης διαδόσεως της διάρρηξης και της ανάδυσης του σεισμογόνου ρήγματος στην επιφάνεια. Οι σεισμικές αυτές κινήσεις παρουσιάζουν μακροπερίοδο συχνοτικό περιεχόμενο με ακολουθίες παλ-μών στην επιτάχυνση και στην ταχύτητα οι οποίες έχουν δυσμενείς συνέπειες στην απόκριση των μελετούμενων συστημάτων. Χρησιμοποιήθηκαν 120 επιταχυνσιογραφήματα από διεθνείς σεισμούς των τελευταίων δύο δεκαετιών αλλά και μεγάλος αριθμός εξιδανικευμένων παλμών. Τα μη-γραμμικά συστήματα της ερευνάς μας είναι συστήματα τριβής επί οριζοντίου και κεκλιμένου υποβάθρου, στα οποία η ολισθαίνουσα μάζα στηρίζεται σε βάση διεγειρόμενη είτε απευθείας είτε μέσω ενός μονοβάθμιου ταλαντωτή.

ΤΑ ΣΥΣΤΗΜΑΤΑ ΟΛΙΣΘΗΣΗΣ ΣΤΗΝ ΓΕΩΤΕΧΝΙΚΗ ΜΗΧΑΝΙΚΗ

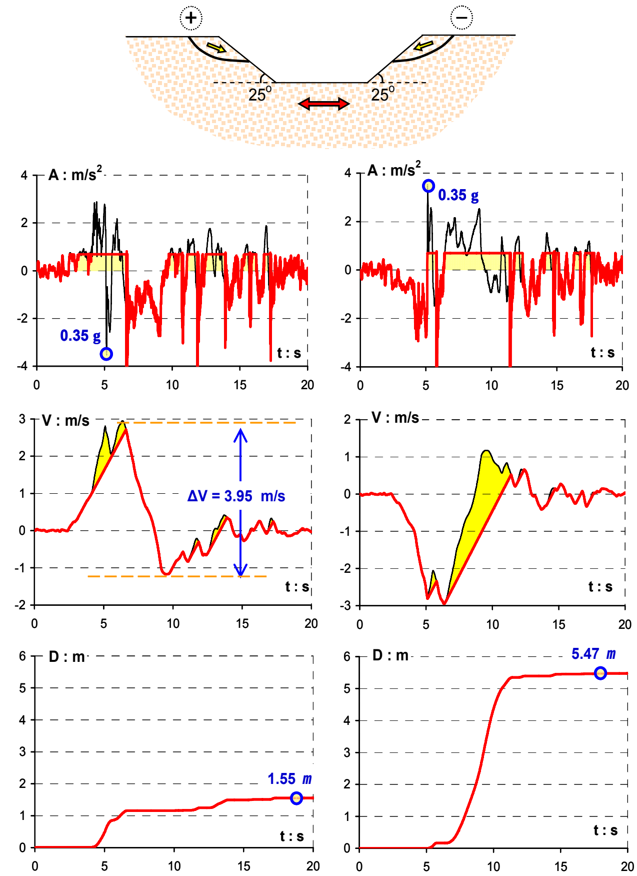
Η δυναμική ολίσθηση στερεού σώματος, μάζας m, επί κεκλιμένου επιπέδου βρίσκει εφαρμογή σε αρκετά προβλήματα της γεωτεχνικής σεισμικής μηχανικής. Ήδη από το 1965 την πρωτο-εισήγαγε ο Newmark για τον υπολογισμό της σεισμικής απόκρισης χωματίνων φραγμάτων. Το σύστημα ολισθαίνοντος σώματος επί κεκλιμένου επιπέδου χρησιμοποιείται σήμερα, μεταξύ άλλων, για την εκτίμηση των δυναμικών καθιζήσεων επιφανειακών θεμελίων, τον υπολογισμό παραμενουσών μετακινήσεων πρανών, και των σεισμικών μετακινήσεων τοίχων αντιστηρίξεως βαρύτητας.

ΒΑΣΙΚΑ ΣΥΜΠΕΡΑΣΜΑΤΑ

1. Τα φαινόμενα της *"κατευθυντικότητας"* και της *"αλτηματικότητας"* που κυριαρχούν εγγύς του ρήγματος προικοδοτούν τους εδαφικούς κραδασμούς με μακροπερίοδους παλμούς στην χρονοϊστορία της επιτάχυνσης ή και της ταχύτητας. Ο ρόλος των παλμών αυτών στην πρόκληση βλαβών σε γεωτεχνικά συστήματα διερευνήθηκε στην διατριβή αυτή μέσω τριών απλών θεμελιωδών συστημάτων τριβής (τύπου Newmark), όπου η καταπόνηση δεν είναι παρά η σεισμικώς προκαλούμενη ολίσθηση ενός στερεού σώματος ή απλού ταλαντωτή.

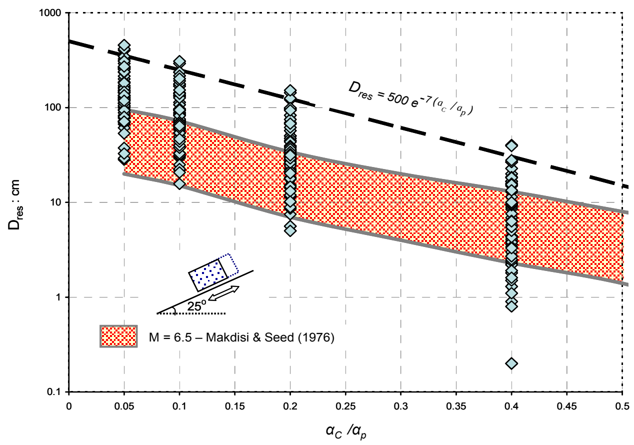
2. Διεγείροντας τα απλά αυτά συστήματα με ένα μεγάλο πλήθος καταγεγραμμένων εδαφικών κραδασμών (επιταχυνσιογραφημάτων) αλλά και με καταλλήλως εξιδανικευμένους παλμούς (wavelets), μελετούμε την επίδραση ενός πλήθους παραγόντων στην ολίσθηση (μέγιστη και τελική). Συμπεραίνουμε ότι χαρακτηριστικά των εγγύς-του-ρήγματος κραδασμών όπως οι μακροπερίοδοι παλμοί στην επιτάχυνση, το μέγιστο βήμα ταχύτητας, η αλληλουχία των παλμών, η ασυμμετρία και πολικότητα της επιτάχυνσης, είναι πολύ σημαντικότερα από τις συνήθεις παραμέτρους όπως η κορυφαία τιμή της επιτάχυνσης και η ταυτόχρονη επιβολή ισχυρής κατακόρυφης διέγερσης.

3. Το σύνολο των λεπτομερών αναλύσεων επέτρεψε την επίτευξη βαθύτερης κατανόησης όχι μόνον του ρόλου των προαναφερθεισών παραμέτρων, αλλά και της απροβλεψιμότητας του μεγέθους της ολίσθησης από έναν συγκεκριμένο κραδασμό. Καθιερωμένα αλλά και νεο-προτεινόμενα "μέτρα εντάσεως" (IM), ή "δείκτες καταστροφικότητας" όπως χαρακτηρίζονται στην διατριβή, είναι από μόνα τους ανεπαρκή για την πρόβλεψη της ολίσθησης — συμμετρικού ή ασύμμετρου συστήματος. Μόνον στατιστικές εκτιμήσεις είναι δυνατές με σχετική αξιοπιστία. Τα καθιερωμένα διαγράμματα Makdisi και Seed αποδεικνύονται ανεπαρκή για την εκτίμηση των ανελαστικών μετακινήσεων γεωτεχνικών συστημάτων από εγγύς του ρήγματος διεγέρσεις (**Σχήμα 1**). Αναπτύσσονται λοιπόν νέες σχέσεις για το άνω πιθανό όριο των ολισθήσεων από τις διεγέρσεις αυτές.



Σχήμα 1: Χρονοιστορίες επιτάχυνσης, ταχύτητας και μετακινήσεις ολισθήσεως δύο ιδεωδώς ταυτόσημων πρανών ευρισκόμενων αντικριστά, όταν υποβάλλονται στην σεισμική διέγερση TCU 068-NS από τον σεισμό του Chi-chi, 1999 στην Ταϊβάν. (a_C/a_H = 0.2 , β = 25 °)

4. Μέσω όλων των παραμετρικών αναλύσεων της διατριβής, αναδύθηκαν συστηματικώς μερικά παράδοξα στην απόκριση του συστήματος σώματος-κεκλιμένου επιπέδου. Ενώ θα περιμέναμε ότι για δύο μαθηματικώς ίδια πρανή ευρισκόμενα το ένα ακριβώς απέναντι στο άλλο, και άρα διεγειρόμενα με την ίδια κίνηση (αλλά αντίθετου προσήμου) να είχαν την ίδια περίπου απόκριση, κάτι τέτοιο ενδέχεται να μήν ισχύει καθόλου όταν η διέγερση έχει την ασυμμετρία των εγγύς-του-ρήγματος επιταχυνσιογραφημάτων! Ένα χαρακτηριστικό παράδειγμα απεικονίζεται στο **Σχήμα 2**: η ολίσθηση που προκαλείται από την καταγραφή TCU 068-NS είναι υπερτριπλάσια απλώς και μόνον αντιστρέφοντας την πολικότητα του επιταχυνσιογραφήματος. Τούτο οφείλεται στον παλμό κατευθυντικότητας της επιτάχυνσης (μεταξύ 5 και 11 s), ο οποίος μεταφράζεται ώς ασύμμετρος παλμός στην ταχύτητα (πλάτους 3.95 m/s). Οι δύο αποκρίσεις, D(+) και D(-), ανάλογα με την πολικότητα της εκάστοτε καταγραφής μπορεί να είναι δραματικώς διαφορετικές (με ποσοστά απόκλισης των μετακινήσεων έως και 400%.



Σχήμα 2: Μετακινήσεις ολισθήσεως (σε εκατοστά) συναρτήσει του λόγου κρίσιμης πρός μέγιστη επιτάχυνση για καταγραφές σεισμών μεγέθους 6.0 < M < 6.9, εν συγκρίσει με τα αποτελέσματα των Makdisi & Seed.

ΜΗ ΓΡΑΜΜΙΚΗ ΑΛΛΗΛΕΠΙΔΡΑΣΗ ΕΔΑΦΟΥΣ – ΚΑΤΑΣΚΕΥΗΣ ΥΠΟ ΙΣΧΥΡΗ ΣΕΙΣΜΙΚΗ ΡΟΠΗ

Μάριος Αποστόλου
Εθνικό Μετσόβιο Πολυτεχνείο
Σχολή Πολιτικών Μηχανικών

Επιβλέπων: **Καθηγητής Γεώργιος Γκαζέτας**

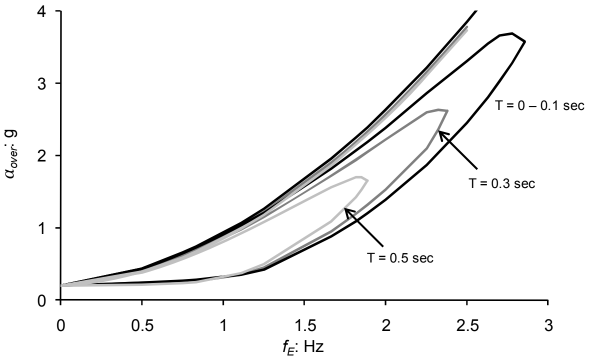
Η διεθνής έρευνα στην σεισμική αλληλεπίδραση εδάφους-κατασκευής τις τελευταίες δεκαετίες έχει ως επί το πλείστον βασισθεί σε δύο θεμελιώδεις παραδοχές: (α) γραμμική (ή ισοδύναμη-γραμμική), ιξωδο-ελαστική εδαφική συμπεριφορά και (β) πλήρη επαφή του θεμελίου με το υποστηρίζον έδαφος. Ωστόσο, σε συστήματα υψίκορμων κατασκευών με επιφανειακή θεμελίωση, ακόμα και υπό καθεστώς μέτριας σεισμικής εξαίτησης, η αναπτυσσόμενη ροπή στο θεμέλιο (λόγω της αδράνειας της ανωδομής) ενδέχεται να οδηγήσει σε εξάντληση της αντοχής της διεπιφάνειας (και κατά συνέπεια σε αποκόλληση του θεμελίου). Επιπλέον, η ανάληψη της αδρανειακής ροπής του θεμελίου από το έδαφος έχει ως αποτέλεσμα την ανάπτυξη σημαντικών πλαστικών ζωνών στο εδαφικό υλικό πλησίον της θεμελίωσης. Από τα παραπάνω αναδεικνύεται η αναγκαιότητα ενδελεχούς αντιμετώπισης των μη-γραμμικών φαινομένων που σχετίζονται με την δυναμική συμπεριφορά συστημάτων επιφανειακού θεμελίου–εδάφους.

Δυναμική απλών συστημάτων λικνισμού: Ως εισαγωγή στην ανάλυση της λικνιστικής συμπεριφοράς υψίκορμων κατασκευών μελετάται η δυναμική δύο τυπικών συστημάτων απλής έδρασης σε ανένδοτη θεμελίωση (πλήρης αποκόλληση και ανασήκωμα του θεμελίου): (α) το απολύτως στερεό σώμα και (β) ο μονοβάθμιος ιξωδο-ελαστικός ταλαντωτής. Για την μελέτη τους καταστρώνονται κατά Lagrange οι μη-γραμμικές εξισώσεις κινήσεως και επιλύονται αριθμητικώς, μέσω της μεθόδου *άμεσης διατύπωσης*. Παράλληλα, υπολογίζεται η απλοποιημένη απόκριση σύμφωνα με την *κατά τμήματα γραμμικοποιημένη* διατύπωση των εξισώσεων κινήσεως. Ο λικνισμός του απολύτως στερεού σώματος μελετάται υπό συνθήκες ελεύθερης και εξαναγκασμένης ταλάντωσης σε αρμονική και σεισμική διέγερση. Η μελέτη της ελεύθερης ταλάντωσης επικεντρώνεται στην *γεωμετρική ανάλυση* του δυναμικού συστήματος. Προκύπτει ότι η οριακή ανατροπή συντελείται σε άπειρο χρόνο και μετά από μία κρούση το πολύ. Στην περίπτωση υψίκορμων σωμάτων, το γραμμικοποιημένο σύστημα επαρκεί για τον υπολογισμό της απόκρισης. Η ανάλυση επεκτείνεται για αρμονική διέγερση. Διερευνώνται και άλλες μορφές μη-γραμμικής αρμονικής ταλάντωσης όπως η υποαρμονική περιοδική και η οιονεί–περιοδική ταλάντωση. Επιπλέον, κατά την μελέτη σε αρμονική διέγερση υπολογίζονται *φάσματα λικνισμού* του μη-γραμμικού και του γραμμικοποιημένου συστήματος.

Για την ανάλυση υπό σεισμική διέγερση χρησιμοποιούνται και πραγματικές καταγραφές σεισμικών επεισοδίων. Πλήθος παραμετρικών αναλύσεων οδηγεί σε κανονικοποιημένα φάσματα. Διαφαίνεται ότι για επαρκώς μεγάλες (ογκώδεις) κατασκευές η απόκριση είναι προβλέψιμη. Στην περίπτωση μάλιστα εξιδανικευμένων παλμικών διεγέρσεων, το πλάτος της γωνίας περιστροφής υπολογίζεται υπό κανονικοποιημένη μορφή μέσω απλών διαγραμμάτων. Από το σύνολο των αποτελεσμάτων προκύπτει ότι το πλάτος της γωνίας λικνισμού είναι ανάλογο *της ραδινότητας* (slenderness) της κατασκευής, της έντασης και κυρίως της δεσπόζουσας περιόδου της σεισμικής διέγερσης. Για τον μονοβάθμιο ελαστικό ταλαντωτή, διερευνάται η αλληλεπίδραση της λικνιστικής στροφής με την καμπτική παραμόρφωση. Μελετώνται δε διεξοδικά οι συνθήκες κατά τις οποίες τα υψηλά επίπεδα λικνισμού οδηγούν τελικώς σε ανατροπή. Ειδικότερα, υπολογίζεται παραμετρικά η ελάχιστη απαιτούμενη σεισμική επιτάχυνση για. Συμπεραίνεται ότι με την αύξηση της ευκαμψίας της ανωδομής συρρικνώνονται προοδευτικά η "περιοχή" της ανατροπής με κρούση και ότι υπάρχει μία κρίσιμη ιδιοπερίοδος, πέραν της οποίας ο μονοβάθμιος ταλαντωτής μπορεί να ανατραπεί μόνον προτού υπάρξει έστω και μία κρούση.

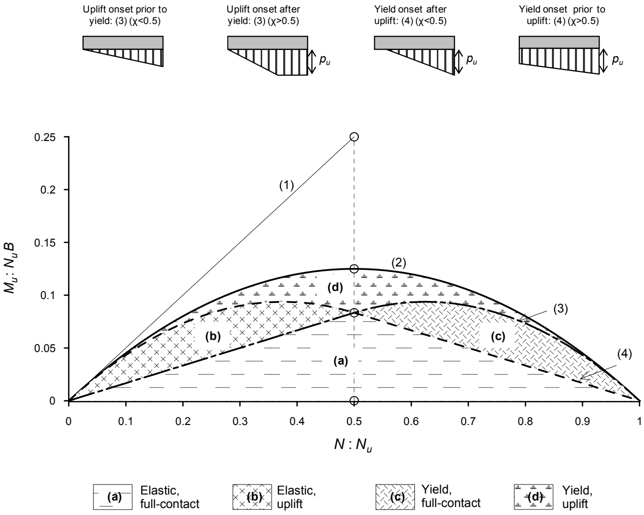
Ανάλυση της λικνιστικής απόκρισης με πεπερασμένα στοιχεία: Η ανάλυση της λικνιστικής απόκρισης επεκτείνεται εισάγοντας την ενδοσιμότητα του εδάφους. Μέσω ενός εξελιγμένου αλγόριθμου επαφής, καθίσταται δυνατή η ρεαλιστική προσομοίωση της λικνιστικής συμπεριφοράς ακόμα και κοντά στα όρια της ανατροπής. Η μη-γραμμική εδαφική συμπεριφορά προσομοιώνεται κυρίως με το κριτήριο διαρροής von Mises και τον ισοτροπικό/κινηματικό νόμο κράτυνσης στην μετελαστική περιοχή. Το προσομοίωμα αυτό θεωρείται κατάλληλο για την ανάλυση της δυναμικής/ανακυκλικής απόκρισης μαλακών αργιλικών σχηματισμών υπό αστράγγιστες συνθήκες. Κατ' αρχάς μελετάται η μονοτονική απόκριση και υπολογίζονται καμπύλες ροπής–στροφής και μετακίνησης–στροφής. Η ανάλυση επεκτείνεται χρησιμοποιώντας σεισμικές διεγέρσεις στο βραχώδες υπόβαθρο. Διαπιστώνεται ότι η κατασκευή μπορεί να εκτελέσει λικνισμό χωρίς αστοχία για επιταχύνσεις πολύ μεγαλύτερες της στατικής επιτάχυνσης ανατροπής. Επιπλέον η γεωμετρικώς μη-γραμμική συμπεριφορά κυρίως (αποκόλληση της διεπιφάνειας) αλλά και η ανελαστική δράση του εδάφους θεμελίω-σης μπορούν να επιδράσουν ευεργετικά στον περιορισμό και το ψαλίδισμα των αναπτυσσομένων αδρανειακών δυνάμεων στην ανωδομή.

Φέρουσα ικανότητα επιφανειακών θεμελίων λόγω μεγάλης σεισμικής ροπής: Η διερεύνηση της σεισμικής συμπεριφοράς των επιφανειακών θεμελίων κοντά στην αστοχία καταδεικνύει ότι η οριακή ροπή ανατροπής επηρεάζεται αφενός μεν από την αποκόλληση του θεμελίου σε περιπτώσεις σχετικώς μεγάλων συντελεστών ασφαλείας έναντι κατακόρυφου φορτίου (FS > 2.5), αφετέρου δε από τις αναπτυσσόμενες πλαστικοποιήσεις στο έδαφος θεμελίωσης για κατακόρυφα φορτία κοντά στο μέγιστο επιτρεπόμενο (1 << FS < 2.5). Υπολογίζονται παραμετρικά αδιάστατοποιημένα διαγράμματα αλληλεπίδρασης των δράσεων (Μορ, Qορ, Nορ) του θεμελίου υπό στατικές και δυναμικές συνθήκες.



Σχήμα 1 Φάσμα ανατροπής μονώροφης κατασκευής (p = 1.7 rad/sec, θ_c = 0.2 rad) με περίοδο T = 0.1, 0.3, και 0.5 sec, σε διέγερση ημιτονικού παλμού συχνότητας f_E. Ο συντελεστής κρούσης είναι 0.89 (ελαστική κρούση).

Ελατηριωτό δυναμικό προσομοίωμα Winkler για την ανάλυση της ανελαστικής λικνιστικής απόκρισης: Αναπτύσσεται απλοποιημένη μεθοδολογία για τον αναλυτικό υπολογισμό των καταστατικών σχέσεων δύναμης–μετακίνησης στο σύστημα θεμελίου-εδάφους μέσω του ελατηριωτού προσομοιώματος. Αντίθετα με την διεθνώς καθιερωμένη προσεγγιστική επίλυση του προβλήματος που περιορίζει το εύρος εφαρμογής της σε πολύ μικρές μετακινήσεις της ανωδομής, στην παρούσα μελέτη λαμβάνεται υπ' όψιν η αναπτυσσόμενη (γεωμετρικής φύσεως) μή-γραμμικότητα του προβλήματος (φαινόμενο P-δ) που λαμβάνει χώρα σε μεγάλες γωνίες λικνισμού. Αναπτύσσονται κλειστές αναλυτικές σχέσεις για την ροπή, την κατακόρυφη μετακίνηση, και το πλάτος του ενεργού (εν επαφή) θεμελίου συναρτήσει της γωνίας λικνισμού, σε ελαστικό και ανελαστικό έδαφος. Οι καμπύλες αυτές ελέγχονται με τα αποτελέσματα της αριθμητικής ανάλυσης. Με παραγώγιση των αναλυτικών σχέσεων ροπής–στροφής υπολογίζονται καμπύλες αστοχίας ροπής–αξονικής.



(1) $\therefore \dfrac{M_{u,rigid}}{N_u B} = \dfrac{\chi}{2}$

(2) $\therefore \dfrac{M_u}{N_u B} = \dfrac{\chi}{2}(1-\chi)$

(3) $\therefore \dfrac{M_{uplift}}{N_u B} = \begin{cases} \dfrac{\chi}{6}, & \chi \le 0.5 \\ (1-\chi)\left(\dfrac{2}{3}\chi - \dfrac{1}{6}\right), & \chi \ge 0.5 \end{cases}$

(4) $\therefore \dfrac{M_y}{N_u B} = \begin{cases} \dfrac{\chi}{6}(3-4\chi), & \chi \le 0.5 \\ \dfrac{1}{6}(1-\chi), & \chi \ge 0.5 \end{cases}$

Σχήμα 2 Καμπύλες αλληλεπίδρασης ροπής – αξονικής εξαχθείσες με το ελατηριωτό ελαστικοπλαστικό προσομοίωμα, στις καταστάσεις οριακής αποκόλλησης, εδαφικής πλαστικοποίησης και αστοχίας (ανατροπής).

ΑΡΘΡΑ

THE STABILITY OF NATURAL AND CUT SLOPES IN STIFF CLAYS [(*)]

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Summary

The stability of natural and cut slopes in stiff and structured clays and the corresponding selection of design parameters are of great interest for infrastructure works. The most widespread method of analysis for design purposes is the limit equilibrium, as it combines some simplicity together with a long time experience. Limit equilibrium analyses require the estimation of the shear strength parameters, i.e. the cohesion, *c*, and the angle of shearing resistance, φ. There are two basic slope failure situations: a) the first – time slides, which include the mobilization of post – peak fully softened strength and b) the reactivated slides, which include the mobilization of residual strength. The appropriate selection of the fully softened strength parameters is of great importance for appropriately interpreting activated slides or analysing slopes. A reference is made to the post – rupture strength, which for the usual range of stresses seems to fall closely to the fully softened strength.

Key words

Cut slope; fully softened; post-rupture; progressive failure; residual; stability; stiff clay; structure

1. INTRODUCTION

The stability of natural and cut slopes in stiff and structured clays and the corresponding selection of design parameters are of great interest for infrastructure works, especially at the Balkan area as there are major civil works either under construction or planning. The most widespread method of analysis for design purposes is the limit equilibrium, as it combines some simplicity together with a long time experience. Limit equilibrium analyses require the estimation of the shear strength parameters, i.e. the cohesion, *c*, and the angle of shearing resistance, φ.

Normally, the following two situations can be examined during the design of natural and cut slopes:

- First time slides, of previously stable slopes that include the following conditions:
  - Short term, which usually correspond to the end of construction state of cut slopes. As pore pressures are not known, the undrained shear strength parameters are typically used, that is undrained cohesion, c_u (or S_u), and undrained angle of shearing resistance, $\varphi_u=0$. It includes total stress analysis.
  - Long term, which usually correspond to the steady state seepage conditions of natural or cut slopes. A phreatic surface is assumed/calculated and drained shear strength parameters are used, that is effective cohesion, c', and effective angle of shearing resistance, φ'. It includes effective stress analysis.
- Reactivated slides, which require the existence of a presheared failure surface activated due to natural or manmade processes and include:
  - Slow rate slides, which involve the use of residual shear strength, that is effective residual cohesion, $c_r'=0$, and effective residual angle of shearing resistance, φ_r'. Sometimes, they are previously creeping slopes.
  - Fast rate slides, which involve the use of fast residual shear strength ($c_r=0$, and φ_r). It includes an influence of the accelerating displacement (i.e. displacement rate) on the shear strength.

An important aspect of both first time and reactivated slides on stiff and structured clays is the progressive nature of failure ([5], [7] [11]). For instance, despite that the design of the long – term stability has been traditionally associated with the use peak strength shear parameters (c_{ps}', φ_{ps}'), early research (e.g. [35], [36]) has exhibited that at the time of failure the mobilized shear strength is lower than the peak. This is assisted by the softening of these clays during pre-failure slow creep shear displacements. The soil structure and the existence of fissures, as well as the high plasticity, enhance the possibility of softening. Moreover, sedimentation, erosion and unloading processes may form weak strength surfaces, which can manifest the overall stability of the slope. In addition, the final triggering of the instability is strongly associated with the pore pressures within the soil or along the slip surface.

The mobilized or operational shear strength is often referred to as the fully softened strength (c_s', φ_s', see [35]), which has been argued that to be close to the critical state strength (φ_{cv}'). The critical state strength ([31][31]) is defined as a condition in which continued shearing takes place at constant shear stress, constant effective stress and constant void's ratio. However, experience shows that the identification of well-defined critical states from laboratory tests on natural overconsolidated clays is usually not possible, as a continuous drop in post-peak strength takes place with continuing dilatancy. Recent research by Burland *et al* ([12]) showed that many stiff natural intact clays exhibit brittle behavior with the formation of slip surfaces or shear bands at peak strength, followed by rapid reduction in strength. The shear and normal stresses acting on the rupture planes have been termed as the post rupture strength.

The post rupture strength envelope of a number of natural clays (see [12]) has been found experimentally to lie closes to the critical state envelope for the reconstituted material. These findings have important implications for evaluating the operational strength of first time slides within stiff clays.

Finally, there are four widely used measures of strength for stiff clays materials: a) peak strength (c_{ps}', φ_{ps}'), which corresponds to maximum shear stress, b) residual strength (c_r', φ_r'), which corresponds to the large strain strength, c) critical state strength (c_{cv}', φ_{cv}'), which corresponds to post – peak constant volumetric strain or excess pore pressures before residual strength and d) post-rupture strength (c_{pr}', φ_{pr}'), which corresponds to the complete formation of post peak rupture plane. Critical state strength has been traditionally determined from reconstituted clays, whereas post – rupture strength is more evident in natural stiff clays, in which strain localization is pronounced. Other useful measures of strength are the maximum angle of frictional resistance (φ_{max}'), which corresponds to maximum stress ratio $(\tau/\sigma_n')_{max}$, and the angle or frictional resistance (φ_{peak}') at peak shear stress (i.e. τ_{peak} or q_{peak}).

2. ELEMENTS OF SOIL BEHAVIOR OF NATURAL STIFF CLAYS

Clays in their natural (in situ) state are generally structured ([10], [25]), that is they have components of strength, stiffness and dilatancy that cannot be accounted for solely by soil's current stress state and void's ratio (*e*, or specific volume: v=1+e). A soil is in a structureless state when its mechanical behavior is completely described by the current stress and void's ratio. A structureless soil is radially compressed ($\eta=q/p'$=constant, where $q=\sigma_1'$- σ_3' and $p=(\sigma_1'+2\sigma_3')/3)$, which reaches a constant dilatancy ($d_q=\dot{\varepsilon}_q/\dot{\varepsilon}_v=(\dot{\varepsilon}_1-\dot{\varepsilon}_3)/(\dot{\varepsilon}_1+2\dot{\varepsilon}_3)$=constant). Upon this condition, soil's state lies on the Intrinsic Compression Curve ([4], [10]), which is shown in Figure 1.

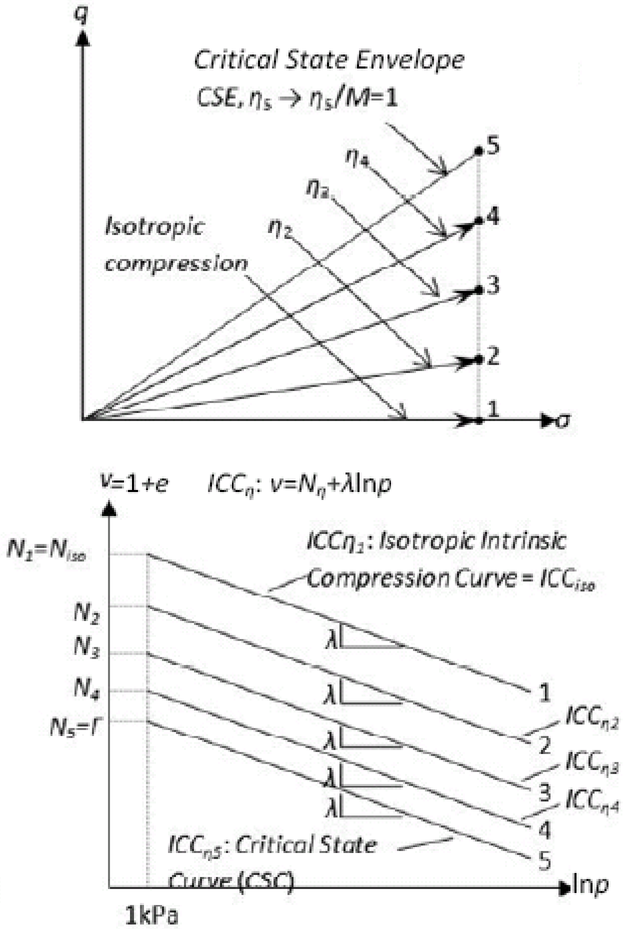


Figure 1. Intrinsic compressibility for linear v–lnσ΄ ICC_η curves ([4])

The mechanical properties of a reconstituted clay are termed intrinsic properties (Burland, 1990), which form a valuable frame for assessing and evaluating the properties of a natural intact clay. The intrinsic compression curves, in an oedometer, for many clays form an almost unique line when replacing the voids' ratio, e, with the voids' index $I_v=(e-e_{100}^*)/(e_{100}^*-e_{1000}^*)$ (Figure 2). This line is termed as the Intrinsic Compression Line (*ICL*).

Many natural normally consolidated soils, laid down under still water conditions, lie upon a well-defined line, namely the Sedimentation Compression Line (*SCL*), which is placed to the right of the *ICL* (Figure 2). When an undisturbed sample of heavily overconsolidated clay is compressed, the compression curve usually crosses the *ICL* and then bends down as appreciable structural breakdown begins ([10], [12]) as shown in Figure 3. In the case of a brittle bonded clay with microstructure the yield state, σ_{vy}, is well to the right of the *SCL* due to strong interparticle bonding.

The framework provided by the intrinsic behavior during compression may be extended to include shearing behavior. Figure 4 shows the state boundary surfaces in normalized plots of t/σ_{ve}^* (or $q/(Mp_e^*)$) against s'/σ_{ve}^* (or p'/p_e^*), where σ_{ve}^* (or p_e^*) is the equivalent intrinsic (Hvorslev's) pressure (or mean stress) for the reconstituted material and t and s' (or q and p') the traditional parameters for the study of the shear strength. The natural or undisturbed (i.e. structured) soil's normalized stress paths (Figure 4b) lay on the right of the corresponding stress paths of the reconstituted soil, gradually approaching the reconstituted soil ones as structural breakdown proceeds.

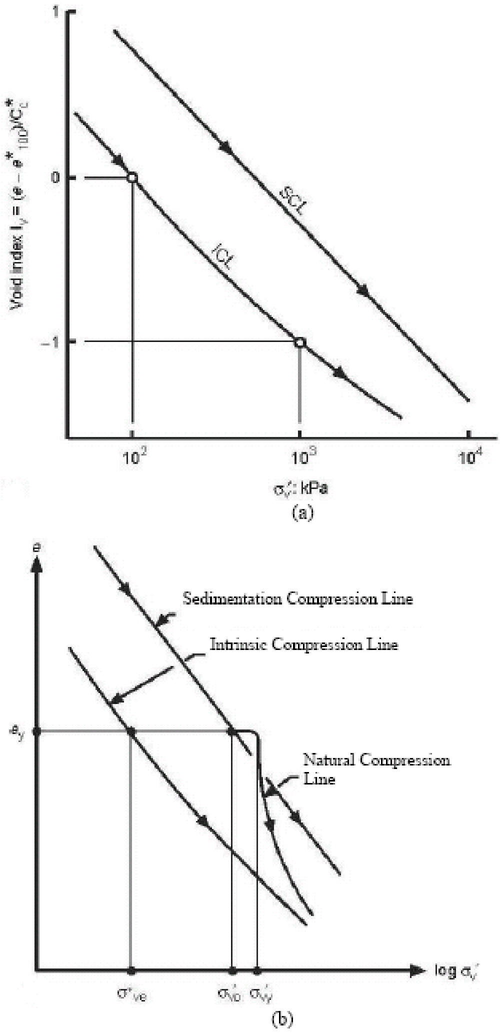


Figure 2. The one dimensional: a) Intrinsic Compression Line, Sedimentation Compression Line and b) Intrinsic Compression Curve, Sedimantation Compression Curve ([10], [12])

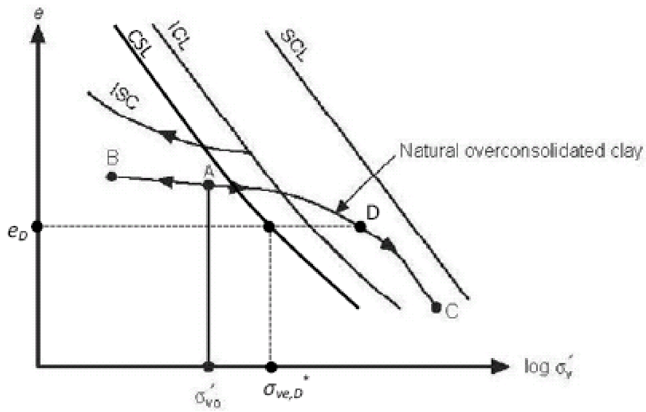


Figure 3. Compression and swell of a natural overconsolidated clay ([10], [12])

Stress history (unloading due to erosion, seasonally fluctuations of water table) and bonding (aging, leaching, cementation, thixotropy, etc) are the most important agents of structure and result in an increased interparticle resistance (i.e. in addition to that offered by the interlocking of soil grains/particles). Figure 5 illustratively shows the effects of structure on the shearing response of unfissured clayey soils. Thus, structured soils when loaded:

- They usually initially develop higher strength, stiffness and dilatancy compared to that of a reconstituted soil with the same history.
- They undergo a destructuration process, which deteriorates the influence of interparticle bonding.
- Upon high strains they reach a fully destructured state.

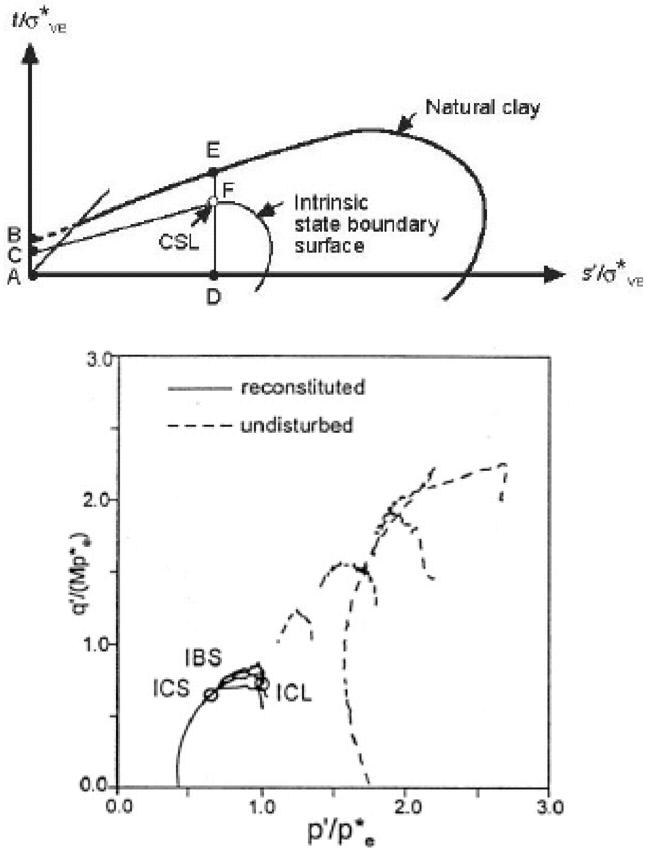


Figure 4. Comparison of natural (undisturbed) state boundary surfaces with the intrinsic (reconstituted soil) ones showing increased resistance to compression and: a) Burland et al ([12]), b) (Cotecchia et al, [17])

Therefore, depending on the initial state and the magnitude of structure, the material can undergo different degrees of softening as a result of destructuring. The rate of the structure deterioration seems to be a combined function of the soil index properties (e.g. *LL*, *PI*, *CF*), the amount of bonding (which can result in yield stresses to the right of the Sedimentation Compression Line, *SCL*, [10]) and the initial state (e.g. to the left or to the right of *SCL*). When a soil state lies to the right of *SCL* it is anticipated that the rate of structure deterioration is rapid. Moreover, it seems that as *PI* increases, the behavior becomes more brittle upon shearing.

The peak strength (c_{ps}', φ_{ps}', Figure 6) of soils corresponds to a maximum shear stress (τ_{max} or q_{max}) that develops during slip formation and is usually applied in design. However, the residual strength (c_r', φ_r', [6], [35]) and the post – rupture strength (c_{pr}', φ_{pr}', [12]) are two important cases of mobilized soil strength along shear surfaces, which affect the stability of slopes. A useful reference state is the critical state ([31]), which is the mobilized strength (φ_{cv}') during a shearing under a constant volume or a constant excess pore pressure.

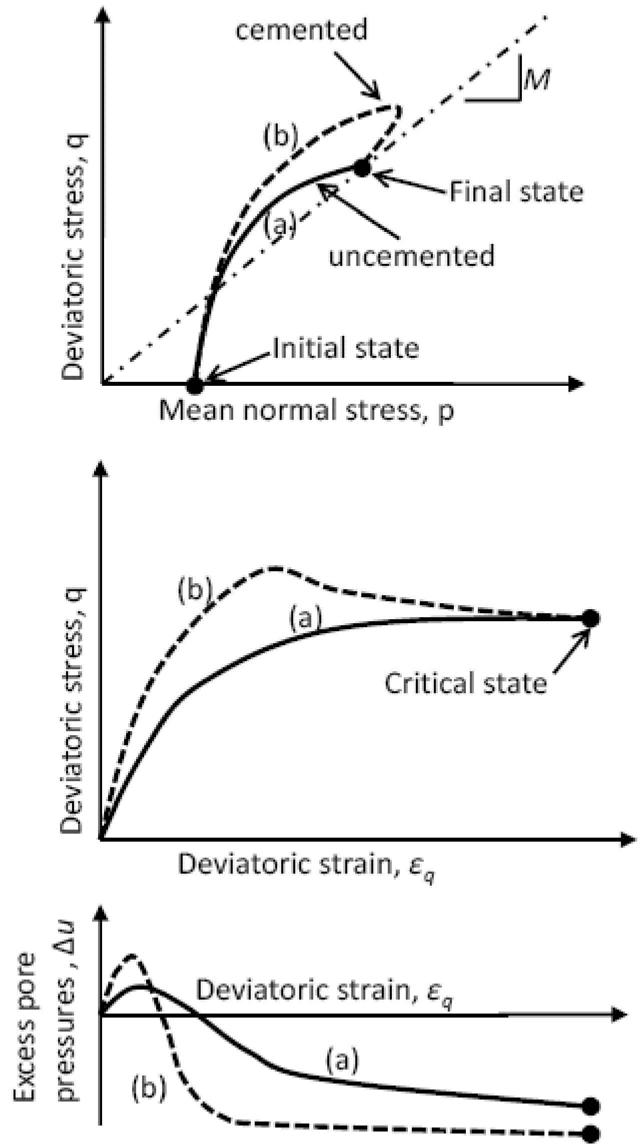


Figure 5. Influence of structure on shearing response (from [2])

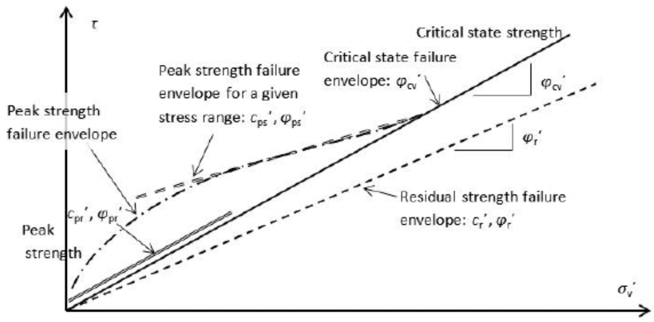


Figure 6. Mohr – Coulomb failure envelopes in soil mechanics.

In addition, stiff overconsollidated clays may mobilize a maximum angle of shearing resistance (φ_{max}') and a corresponding maximum stress ratio ($\eta_{max}=(q/p')_{max}$) before peak shear strength, i.e. it does not represent a failure state. For these clays it is $\varphi_{max}' > \varphi_{cv}'$ while $\eta_{max}>\eta_{peak}>M$, where M is the critical slope. As already mentioned, peak strength (c_{ps}', φ_{ps}'), critical state strength (φ_{cv}'), post – rupture strength (c_{pr}', φ_{pr}') and residual strength (c_r', φ_r') parameters are commonly used for analyses purposes. Moreover, the existence of fissures may have a dominant role on the overall response, reducing greatly the available strength close to residual.

Concerning the stability of slopes, there is evidence that the combined effect of the gradual structure deterioration and the progressive development of shear bands largely influence the stability of first – time slides, which usually trigger at a post-peak mobilized strength (e.g. [36], see next paragraph). Concerning the stability of reactivated slides, it seems that the large strain strength (i.e. residual) has a dominant role. It is normal to account for the effect of pore pressures developments and creep displacements accumulation.

3. FIRST – TIME SLIDES

A first – time slide may develop in either a natural or a cut slope. Concerning the cut slopes in clays, stability may be critical:

- immediately after excavation (end of construction state), which means that excess pore pressures have just started to dissipate and is commonly referred to as the "undrained conditions" or
- a long time after construction, when pore pressures have equilibrate and no excess pore pressures exist, which corresponds to steady state flow conditions and is commonly referred to as the "drained conditions".

The "undrained conditions" are more likely to be more critical for soft clays, while the drained conditions are more likely to be more critical for stiff clays (Figure 7 and [5], [8], [32]). Our present understanding of long term slope behavior in stiff clays relies on field studies of slope failures together with their back analyses. Back analyses of natural or excavated slope failures have shown that the average shear strength along the slip surface may be less than the peak value as a consequence of two processes: time – dependent softening and strain – dependent weakening.

The time dependent softening is the process of the decrease in shear strength of a mass of fissured clay as consequence of environmental factors, associated with unloading over a length of time. Important factors that enhance the softening are the structure development and the existence of fissures, as well as high plasticity. Sedimentation, erosion and unloading processes may form weak strength surfaces, which may manifest the overall stability of the slope. Concerning the triggering of the first time slides in stiff clays, this is strongly associated with the long term process pore pressures recovery ([14], [37], [42]), assisted by creep generated positive pore pressures.

The strain dependent weakening occurs in soils with brittle behavior, i.e. soils with a significant difference between peak and residual strength. This is associated with the progressive failure mechanism, which is accompanied by relatively large displacements along the slip surface. This weakening is also assisted by pre-failure slow rate creep shear displacements.

Both time – dependent softening and strain – dependent weakening are directly related with the progressive type of failure, often called as delayed failure, of slopes in stiff clays.

3.1. The progressive failure mechanism – delayed failure

As natural stiff clays are usually overconsolidated due to surface erosion or melting of ice caps, the unloading due to natural processes may result in the development of some macro – fissures of low strength. The same can happen in cut – slopes, which gradually results in deterioration of micro – structure and shear strength along the macro – fissures. This softening is enhanced by processes such as seasonal piezometric level variations and freeze – thaw cycles.

Immediately after an excavation or erosion, the total stresses within the soil body reduce and, as the behavior is practically undrained, there is a sudden drop of pore water pressures (see Figure 7). This results in some swelling accompanied by softening due to extension of the fissures and by differential shear displacements along the fissure ([28]). It has been already mentioned that this situation is not necessarily critical.

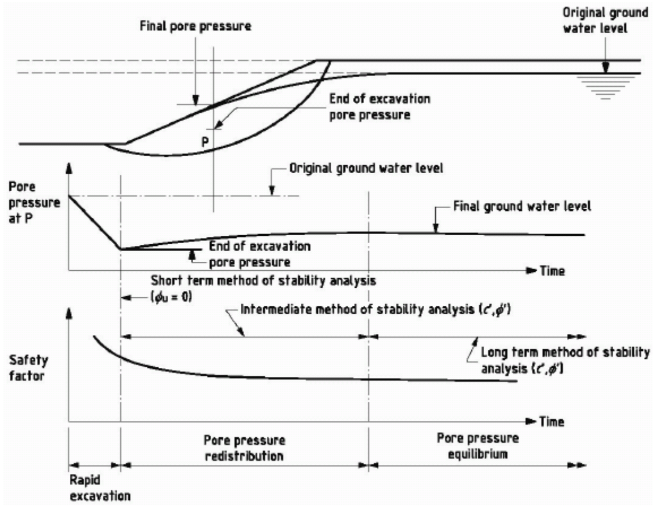


Figure7. Factor of safety development for cut slopes on stiff clays (BS 6031: 2009)

Next, pore water pressures rise towards steady state seepage, while seasonal fluctuations, especially after periods of intense rainfalls, have an important role on the piezometric level. For clayey cut slopes it may take more than 50 years for steady state pore pressures to develop ([14], [37], [42]). Water content near the slip surface is usually greater (see [35]), which is explained by the dilation and consequent water absorption during the rupture of stiff clays. This enhances the softening.

It has been observed in the field that the shear strains along the slip surface are not uniform prior failure (e.g. [11]) and progress mainly from the toe towards the crest. Such a procedure has also been simulated by finite elements analyses ([30]). Therefore, just before failure part of the slip surface is in a post – peak strength state but far from residual. That means that the average shear strength along the slip surface is generally lower than the peak strength and greater than the residual strength (see [5], [15], [35], [37]). At a point the average available strength along the potential slip surface is critical, so that a further reduce of available strength, due to softening, weathering or pore pressure build up, results in a development of a slide.

This is the progressive failure mechanism of stiff clayey slopes (e.g. [5], [11]) and is accompanied by a high piezometric level, not always under steady state conditions (see [42]).

3.2. Mobilized shear strength for long – term stability based on limit equilibrium analyses

The progressive failure mechanism described above means that for limit equilibrium analyses we can only speak for an average mobilized or operative strength along the slip surface. Back analyses of first – time slides (e.g. [15], [28], [35], [36]) have exhibited that at the time of failure (where the factor of safety should be FS=1.0): a) the operating cohesion is significantly smaller than that of the peak strength and close to zero (i.e. $c_{mob}'=0$) and b) the angle of frictional resistance is somewhere between the peak and the critical state value. Figure 8 shows the failure envelopes of some back analyses results from Chandler & Skempton ([15]), which show that a small operative cohesion is present at the time of the triggering.

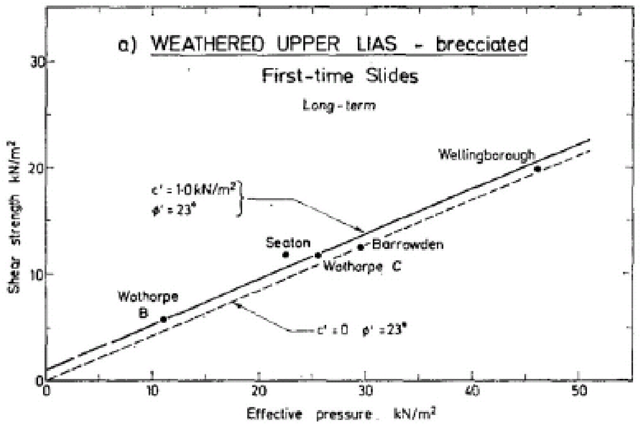


Figure8. Back analyses of first time slides (Chandler and Skempton, 1974)

Skempton ([36]) introduced the concept of "fully softened strength" (c_s', φ_s'), in which the operative or mobilized strength is $c_{mob}'=0$ and $\varphi_{mob}'=\varphi_s'$ (Figure 9). He proposed that: a) for nonfissured stiff clays it is $\varphi_s'\approx\varphi_{ps}'$ to $\varphi_s'\approx\varphi_{NC}'\approx\varphi_{cv}'$, while there can be an appreciable component of cohesion and b) for fissured stiff clays it is $\varphi_s'\approx\varphi_{NC}'\approx\varphi_{cv}'$. The $\varphi_s'\approx\varphi_{NC}'\approx\varphi_{cv}'$ is the "fully softened state".

On the same line of thought Chandler & Skempton ([15]) came to the conclusion that, for high plasticity stiff clays with high *CF*, the $c_{mob}'=0$ and $\varphi_{mob}'=\varphi_s'$ and *FS*=1 concept can lead to a very conservative design. They proposed that a small cohesion intercept must be used. Although it was not clearly stated, they meant that it is conservative when used for redesign, unless the actual slip surface is used. It should be noted that fissured stiff clays have even more pronounced brittleness. Chandler & Skempton also presented a back analysis of a non-brittle, non-fissured boulder clay, which gave an operative c=9kPa. These evidences are close to the laboratory observations about the post-rupture strength (e.g. [12]).

Therefore:

- Non-fissured stiff clays of low to medium plasticity, such as glacial clays or homogenous soft to firm clays of low to medium plasticity, present a mobilized shear strength equal to or greater than the fully softened strength and in some cases (presheared zones) near the intact peak strength ([15], [28], [36]).
- Fissured stiff clays of medium to high plasticity, present a mobilized shear strength that is close to the fully softened strength and in some cases (pre-sheared zones) near the residual strength ([15], [28], [36]).

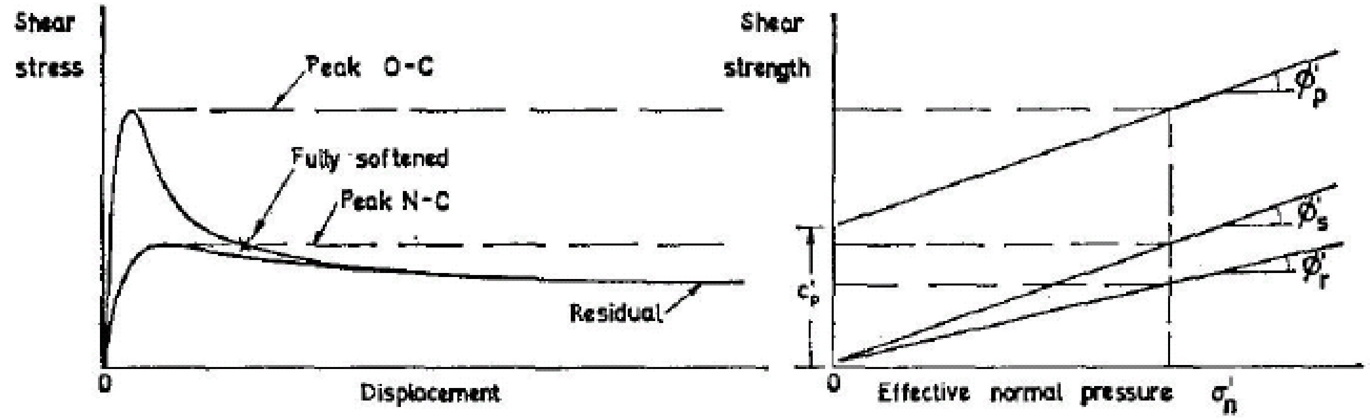


Figure 9. The "fully softened strength" (Skempton, 1970)

The mobilized shear strength strongly depends on whether part of the slip surface moves along bedding or fissures, which results in mobilized angle of shearing resistance lower than the fully softened one ($\varphi_{mob}'<\varphi_s'$). This is more pronounced for high plasticity clays, in which polished and slicken sided shear surfaces may develop, resulting in almost residual mobilized angle of shearing resistance along these surfaces.

It has been argued that the fully softened strength, φ_s', corresponds to a relatively random arrangement of particles with predominant edge to face interaction and interference ([27], [28]). On the contrary, the residual strength condition, φ_r, corresponds to fully oriented platy particles along the slip movement.

It generally is $\varphi_s'\geq\varphi_r'$ and the difference between φ_s' and φ_r' (Figure 10): a) approaches zero at very low plasticity, where particle reorientation is not a factor, b) maximizes at a plasticity index around 50% and c) becomes constant at very high plasticity, where the predominant particle interaction even for a random fabric is face to face ([27]). Mesri & Shahien ([28]) give some empirical relationships about the secant angles of shearing resistance. Moreover, φ_{peak}', φ_s' and φ_r' values are non – linear with respect to normal stress (e.g. Figure 11), which is also in agreement with the experimental observations of Burland *et al* ([12]) for post – rupture strength, but this non-linearity can be ignored, at the moment, for design purposes.

The mobilized strength $\tau=\tau_f-R_1(\tau_f-\tau_r)$ can be expressed by Skempton's residual factor $R_1=(\tau_f-\tau)/(\tau_f-\tau_r)$ ([35]), which, according to the progressive failure mechanism, cannot be constant along the slip surface (e.g [5]). Bjerrum ([7]) proposed the brittleness index $I_B=(\tau_f-\tau_r)/\tau_f$, where τ_f the peak strength and τ_r the residual strength (Figure 12), to describe the effect of brittleness on the progressive failure mechanism. The greater the value of the index is, the more pronounced the progressive failure it is, as it happens in high plasticity stiff clays (e.g. PI>40%). The actual value of φ_{mob}' along the slip surface depends on the degree of softening and on the duration of shearing.

3.3. The influence of pore pressures

The time of the triggering largely depends on the slope geometry and soil's strength and permeability parameters. Chandler and Skempton ([15]) and Skempton ([37]) based mainly on back analyses suggested that upon first – failure it is ru=0.25 – 0.35 (e.g. Figure 13), where $r_u=u/\sigma_v$, but there was evidence that it could be a bit lower sometimes.

The back analyses of Mesri & Shahien ([28]) report r_u values greater than 0.23.

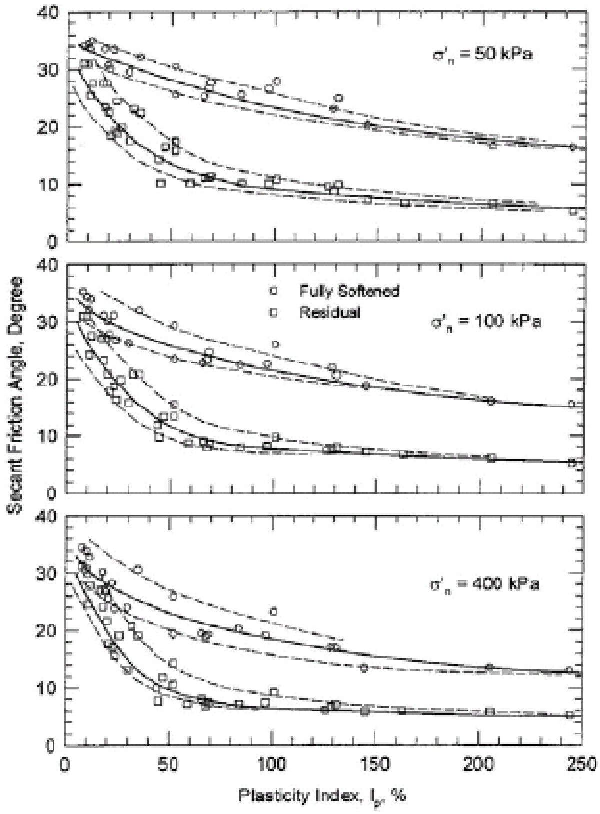


Figure 10. Correlation of fully softened angle of shearing resistance with plasticity index for various stress levels ([28])

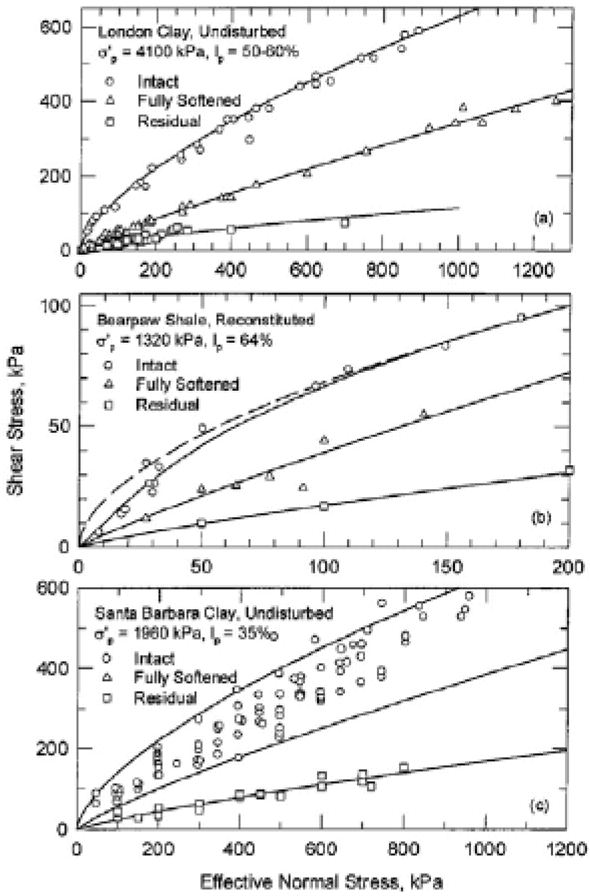


Figure 11. Peak, fully softened and residual strength of some stiff clays ([28]))

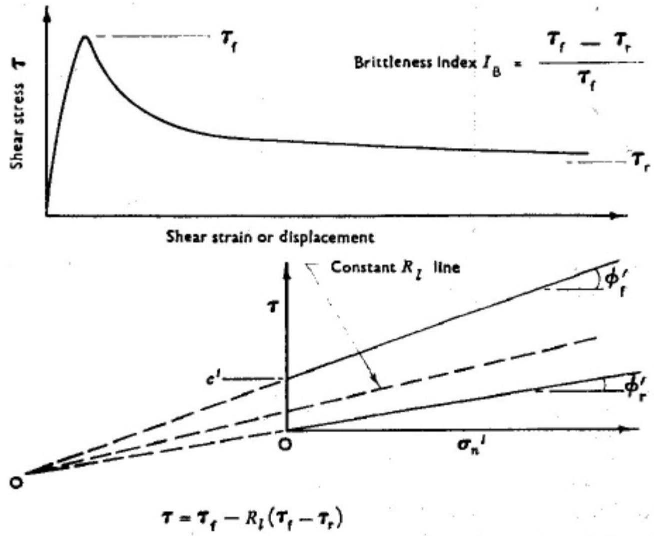


Figure 12. The brittleness index (reprinted from Bishop, 1971)

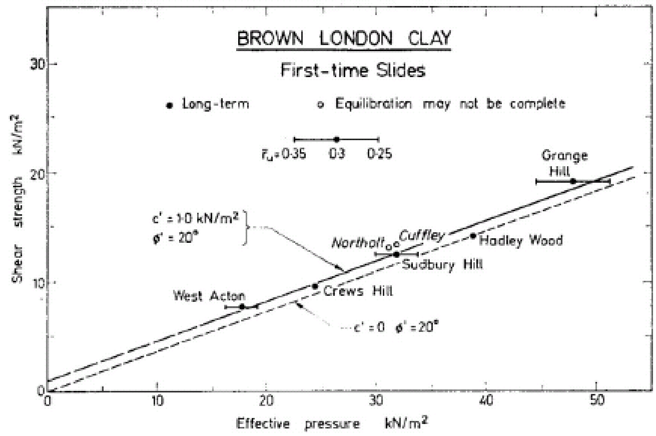


Figure 13. The influence of ru on the back analyses of first time slides ([15])

For an infinite slope of angle a, a flow parallel to the free surface gives a pore pressure of $u=\gamma_w(z-z_w)\cos^2 a$, where z_w the depth of the phreatic surface and therefore it is $r_u=\gamma_w(z-z_w)\cos^2 a/(z)\approx 0.5(z-z_w)\cos^2 a/z$, where z is the depth of the slip surface. For $z_w=0$, it becomes $ru=\gamma_w\cos^2 a/\gamma\approx 0.5\cos^2 a$, which is shown in Figure 14, relating the influence of the slope inclination to r_u.

Chandler and Skempton ([15]) concluded that "provided conservative estimates of r_u are made, based on site investigation, it therefore seems that design of permanent cutting slopes in brown London Clay and weathered Upper Lias Clay, can be derived from the circular arc analysis with c'=1.5 – 2.5kPa (i.e. a small operative cohesion) and $\varphi_{mob}'=\varphi_{ps}'$ with FS=1.0". It could be therefore argued that the slope stability should be checked for a range of possible r_u values.

3.4. The influence of pre-sheared zones

There are cases where pre-sheared zones of residual strength or close to residual strength have an influential role in long term stability of cut slopes. These weak strength zones may become a part of an overall composite slip surface during failure and may assist the progressive failure mechanism of the unsheared stiff clay, reducing the operating strength.

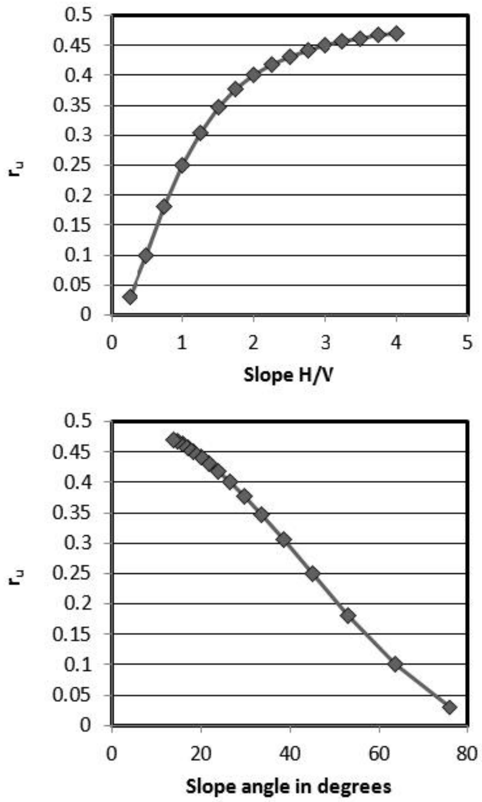


Figure 14. The dependence of ru on slope angle for an infinite slope with a flow parallel to slope and phreatic level on ground surface.

3.5. The influence of creep

Creep displacement has a twofold influence in stiff clays. Firstly, it induces strain softening as a result of their brittle behavior. Secondly, it results in accumulated positive excess pore pressures during the first stages of the creep phenomenon. Both factors shift the soil's state within the slope gradually towards the failure envelope, resulting therefore to a potentially unstable slope depending on the amount of creep.

3.6. Slip surface shape

Many of the delayed first –time slides in stiff clays are deep seated (e.g. [11]), while the field evidence shows that they may be non – circular. This has also been portrayed by the progressive failure Finite Elements Analyses ([30]), which exhibited that for stiff clays (with higher K_o) the rupture surface goes deeper, the progressive failure mechanism is more pronounced and the average mobilized shear strength decreases (Figure 15). The slip surface starts to develop from the bottom progressing upwards ([11], [30]).

The field evidence and the FEA analyses show that for the deep seated surface the slip surface seems not circular. Therefore, it can be argued that concerning the Limit Equilibrium Analyses, it is more appropriate to perform a non – circular critical surface search.

3.7. Experimental post rupture strength

Stiff high plasticity clays typically exhibit brittle post – peak behavior (see [5], [11], [12]) accompanied by a small displacement along the rupture, which can be of the order of 1% of the length of the slip surface ([5]). This rapid loss of strength is more pronounced in fissured clays, which exhibit more brittle behavior than the homogeneous clays, and in high plasticity clays, possibly as a result of the reorientation of the clayey particles and the formation of thin slip surfaces.

Post rupture strength ($c_{pr}'\approx 0$, φ_{pr}') corresponds to the post-peak strength that develops upon full formation of shear bands, usually accompanied by a rapid loss of strength and rigid body motion within a few millimeters ([12], Figure 16). This has been attributed to microstructure deterioration processes. The rapid loss of strength of stiff clays has been depicted from various studies (e.g. [11], [12], [21], [35]).

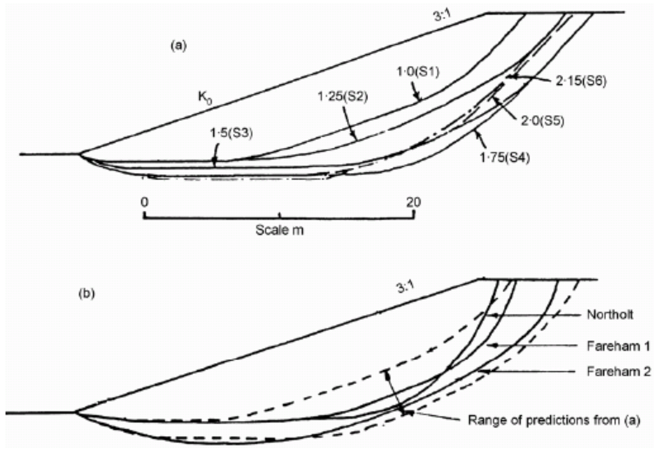


Figure 15. (a) Rupture surfaces predicted by FEA for varying Ko and (b) comparison with some field observations ([30])

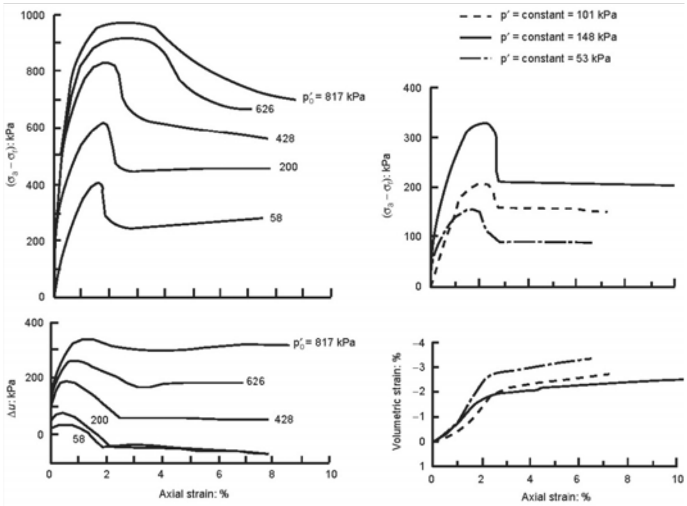


Figure 16. Vallericca clay undrained and drained typical triaxial shear behavior ([12])

The time of the initiation of shear band formation (i.e. strain localization) is not very clear and there are various views. It has been stated that the initiation takes place:

- at about maximum stress ratio, η_{max}, or angle of shearing resistance, φ_{max}, (i.e. before peak) and the slip surface develops up to peak strength (e.g. [1], [43]) or
- at post peak strength and slip surface forms before residual strength ([12], [21]) at about the onset of rapid loss of strength.

During the rapid loss of strength the pore pressures, u, stabilize at the point of the post rupture strength ([12]). This drop of strength occurred in the laboratory over a displacement of less than 0.5mm or strain of less than 1% ([12], [21]). Similarly, Burland *et al* ([11]) examining a slope failure in stiff Oxford clay reported that the shear box tests exhibited a rapid loss of strength near residual for a displacement of 3 to 4mm.

The post rupture strength (c_{pr}', φ_{pr}') develops a bit before critical state strength ($c_{cv}'=0$, φ_{cv}').

Burland *et al* ([12]) report that at post rupture strength there is a small cohesion, while $\varphi_{pr}' \approx \varphi_{cv}'$ (e.g. Figure 17). More specifically:

- At low confining stresses: $c_{pr}'=0$ to 10 kPa, φ_{pr}' is slightly higher than φ_{cv}', but for all practical purposes they propose $\varphi_{pr}'=\varphi_{cv}'$.

- At low to moderate confining stresses: $\varphi_{pr}'=\varphi_{cv}'$.

- At high confining stresses: φ_{pr}' is slightly lower than φ_{cv}'.

The greater strength reduction at higher confining stresses has been attributed to enhanced particle orientation and particle bonds breakage.

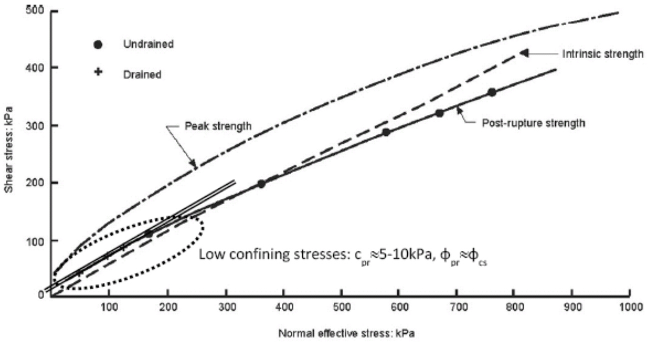


Figure 17. Vallericca clay post rupture strengths compared with intact and intrinsic Mohr – Coulomb [12], modified)

3.8. Experimental post rupture strength

The progressive failure mechanism is less pronounced for low plasticity clays as the difference between φ_s' and φ_r', is lower than in high plasticity clays. Post rupture strength could be associated to the progressive failure of slopes, as there are some similarities on the developed mechanisms. The progressive failure mechanism assisted by soil's brittleness has an important role on the sudden – unexpected triggering of some cut slopes.

At the time of triggering the pore pressures may correspond to ru of the order of 0.25 to 0.35, the triggering can happen some decades after the formation of a cut slope and the slip surface is usually not circular (see also Figure 15).

The fully softened strength is an average mobilized strength along the slip surface of unstable slope and seems to be not far from the post rupture strength. Based on past evidence and results, a value of $c_s'=0$ to 10 kPa and $\varphi_s' = \varphi_{cv}'$ on the developed slip surface looks reasonable.

4. REACTIVATED LANDSLIDES

Reactivated landslides mobilize on pre-sheared surfaces, i.e. surfaces that have experienced shear displacements in the past, developing, therefore, post – peak residual strength (c_r', φ_r'). This is often the case of the stability of natural slopes (e.g. [19], [35]), while another situation is the final equilibrium of first – time slope failures. Both cases may need remediation measures, which usually includes analyses with residual strength parameters.

The identification of an existing slip surface by the geotechnical investigation greatly influences the analyses and the design of a project, which must include a careful selection of the appropriate failure mechanisms and the corresponding shear strength parameters. Moreover, the stability of reactivated slides is very sensitive to small variations of pore pressures (e.g. [19]), which have to be taken into consideration in the analyses.

The residual strength (c_r', φ_r') corresponds to the strength mobilized along a slip surface that undergoes large strains (see [5], [6], [26], [41]) and is the minimum strength that a soil can develop (Figure 6, Mohr - Coulomb envelopes). The strain rate may greatly influence the developed residual strength (Figure 18) and therefore affect the stability. Lemos et al ([24]) have distinguished the slow and the fast residual strength.

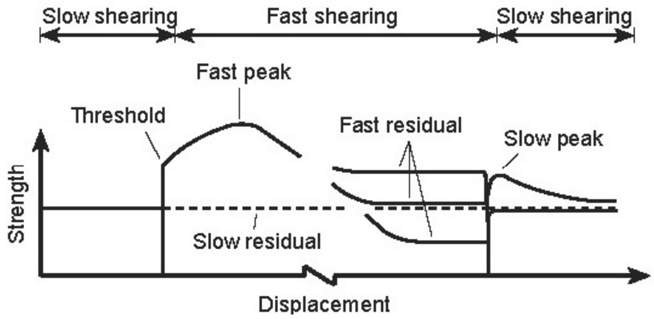


Figure18. Summary of the rate-dependent phenomena for residual strength ([41])

4.1. Mobilized shear strength based on limit equilibrium analyses

Back analyses of reactivated landslides (e.g. [15], [37]) have shown than a small cohesion (i.e. $c_r'=1$ to 2kPa) often develops, which, as it will be shown below, has also been supported by experimental evidence ([6], [15], [26]). This can also justify the delayed triggering of some old landslides, which often exhibit shear creep prior triggering. The back analyses by Mesri & Shahien ([28]) support the non-linear relationship of φ_r' with normal stress, but for design purposes it is usually assumed linear. Moreover, the back calculated residual shear strength parameters are extremely sensitive to the assumed phreatic surface (e.g. [19]).

4.2. Experimental slow residual strength

Slow residual strength is best estimated from ring shear apparatus but reversal shear box tests can also be used. Bishop et al ([6]) report that ring shear apparatus gives lower φ_r values. Although it is typically assumed that $c_r' \approx 0$, there is some evidence that a small cohesion intercept can be present as c_r can vary from 0 to 8kPa (see [26]). This could also be the effect of the non-linear relationship of φ_r' with the normal stress.

The soil particles shape and the angle of interparticle friction, φ_μ, control the mode of shearing and affect the corresponding mobilized residual strength ([26], [35]). In particular, Lupini et al ([26]) identified the following three main shearing mechanisms:

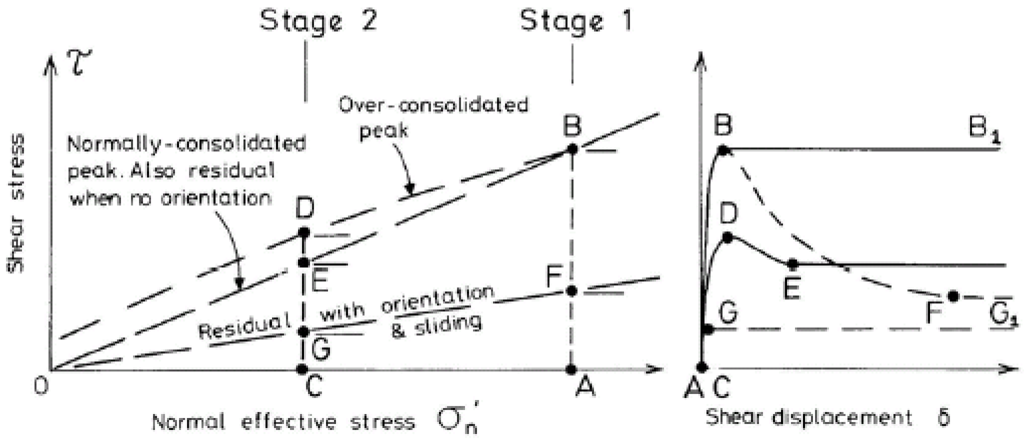
- Turbulent shear, when rotund particles dominate (e.g. sandy silts) and shearing involves rotation and rolling of the particles. A shear zone develops, which results in a $\varphi_r' \approx \varphi_{cv}'$. Lupini et al ([26]) report that also needle-shaped high friction clay minerals (e.g. halloysites) seem to behave as rotund particles.

- Sliding shear, when platy – low friction particles dominate (e.g. the montmorillonite) and shearing involves orientation of soil particles parallel to the direction of shearing giving the lowest possible angle. A permanent polished – slickensided thin slip surface forms, which results in the lowest possible friction $\varphi_r'=f(\varphi_\mu)<\varphi_{cv}'$ that is close to φ_μ.

- Transitional shear, when there is no dominant particle shape and a mixed turbulent and sliding shear is present depending on the distribution of rotund and platy particles within the soil mass. This mechanism is expected to give intermediate residual angle of shearing resistance.

Lupini et al ([26]), based on experimental evidence, proposed the ideal stress – strain response of Figure 19 concerning the ring shear tests. The idealized shear paths CDE and ABB1, of OC and NC clays respectively, correspond to critical state theory behavior, which does not model the effect of the preferred particle orientation on the angle of shearing resistance. This is the pure turbulent shear behavior. Supposing that the clay particles suffer orientation the idealized shear paths CGG1 and ABF, of OC and NC clays respectively, can develop. Both shear paths exhibit a low residual angle of shearing resistance, yet NC clays appear more brittle from peak to residual strength. As clay fraction increases the behavior becomes more.

Experimental results ([6], [22], [26], [27], [35], [40]) show that for high plasticity clays with PI≥40% or CF≥40% the residual angle of shearing resistance can be as low as 10°, even a bit lower (Figures 20, 21). This is due to the sliding shear mechanism. In particular, Lupini *et al* ([26]) report that for natural clayey soils it is $\varphi_r' \approx 5^o$ to 20°. This has been included in BS 6301:2009 "Code of Practice for Earthworks".



Stage	Shear behaviour	
	No orientation	Orientation
1. Normally consolidated; strain hardening	Turbulent shear; non-brittle; high friction angle	Sliding shear; brittle; low residual friction angle
2. Over consolidated	Brittle and dilatant; residual friction angle same as stage 1	Non-brittle; friction angle same as stage 1 residual

Figure 19. Idealized residual behavior according to the ring shear test ([26])

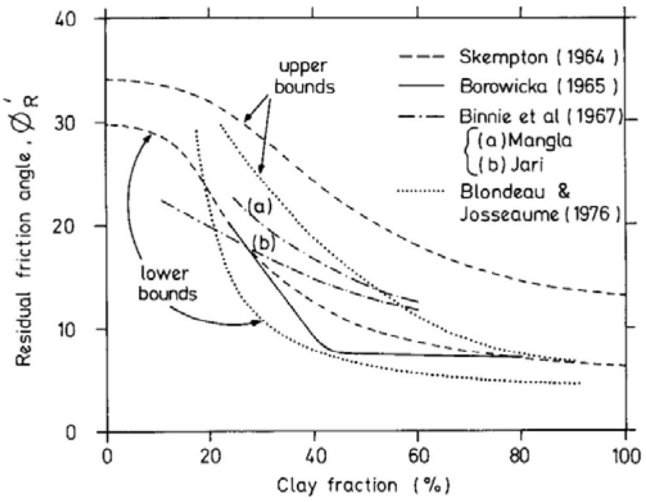


Figure 20. Residual strength as a function of CF (Lupini et al, 1981)

Despite the evidence that φ_r' depends on the effective normal stress (e.g. [6], [22], [28], [40]), for all practical and design purposes it may be considered constant.

Skempton ([38]) and Popescu ([29]) summarizing φ_r' values from different test methods report that:

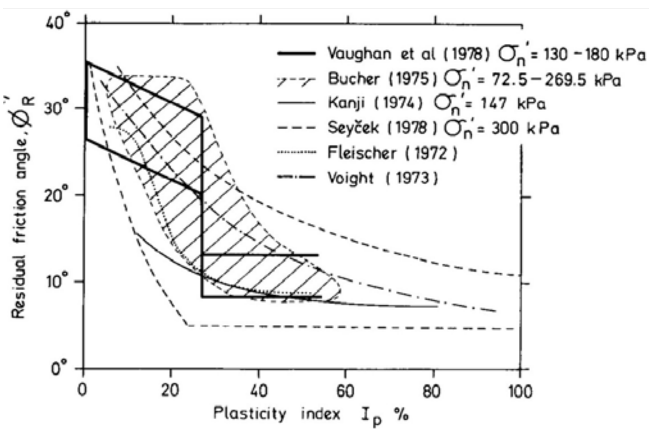


Figure 21. Residual strength as a function of Ip (Lupini et al, 1981)

- direct shear tests on slide plane or bedding plane shear are the most reliable indicator of field residual strength,
- ring shear devices either underestimate by 1° to 2° or approximate the field residual and

- multiple reversal direct shear on clays will probably over-estimate field residual by 1° to 2°.

Skempas ([33]) and Skempas and Chandler ([34]) presented the cut-thin sample technique in the direct shear box, which consists of a slow direct shear on a consolidated 4mm thick sample, pre-cut by a stainless steel wire. This method can give equivalent results to the ring shear device.

4.3. Experimental fast residual strength

Lemos *et al* ([26]) have observed that for shear surfaces which are at a residual state due to slow drained shearing, a fast displacement re-shearing can lead to the responses presented in Figure 18. These three responses correspond to neutral ($\varphi_{r,fast} \approx \varphi_{r,slow}$), positive ($\varphi_{r,fast} > \varphi_{r,slow}$) and negative ($\varphi_{r,fast} < \varphi_{r,slow}$) rate effects (see [41]). Tika *et al* ([41]) reported that mostly a negative rate effect is more pronounced in soils of low to intermediate clay fractions (CF = 3 to 5%) and low to high plasticity (PI=10 to 37%). These are soils mainly of turbulent and transitional shear mechanism. Soils of sliding shear mechanism may exhibit either a negative or a positive rate effect (Figure 22). They associated this effect with the increase of the water content and a corresponding dilation of the shear zone.

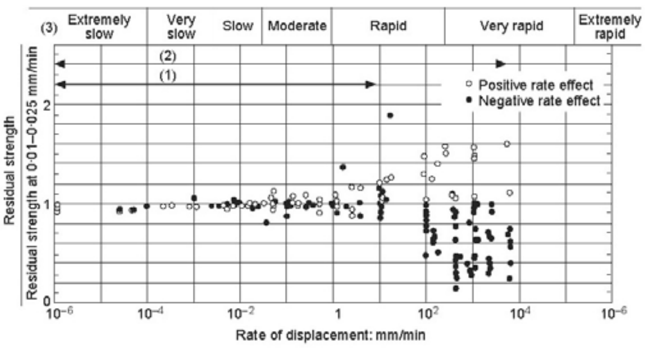


Figure 22. Variation of residual strength of cohesive soils with rate of displacement (Tika et al , 1996)

4.4. Implications on slope stability analyses

The residual strength has been associated with:

- First-time slides: at the final equilibrium state the shear strength along the slip surface is residual as large shear strains have taken place. Tika *et al* ([41]) claim that if the displacement rate effect on the residual strength is positive, the slide moves slowly, while for a negative effect it moves fast.

- Reactivated landslides: they are in a limit equilibrium state with constant strength (a small cohesion may also be present), since the slow residual strength does not change with displacement. For these cases a negative effect of the displacement rate may lead to catastrophic velocities and large displacements ([41]). Often a human intervention provides the situation for reactivation, which is also strongly associated with piezometric level along the slip surface. Typical examples are the Malakassa landslide ([23]) and the Tsakona landslide (Figure 23, [19], [39]). Slow creep displacements may result in strain softening (however it is not brittle) and on pore pressure accumulation.

The rate effect can be important on seismically loaded slopes, especially for reactivated slides. Moreover, small variations of the phreatic surface may critically influence the stability of reactivated slides.

5. DESIGN FACTORS OF SAFETY FOR LIMIT EQUILIBRIUM ANALYSES

Concerning the operational strength for design of a natural or a cut slope on stiff clay, one has to answer the question: is there any zone of weakness within the soil mass? This investigation includes any pre-existing slip surface, fissuring and weathering zone and is a requirement of BS 6031:2009 "Code of Practice for Earthworks" (see paragraphs 6.4.1.1 and 7.2.2 of BS 6031, [9]). If so, reduced shear strength should be considered along these zones. For instance, a reactivation mechanism should include residual strength upon the slip surface. The shape and extend of any such surface greatly influences the overall stability. The displacement rate effect on residual strength may also be considered, especially for seismic loading. On the other hand, a progressive failure check of first – time slides is usually not taken into account, with an exception of the guidelines of BS 6031: 2009, which recognizes the long term steady state condition as the most critical one (Figure 7).

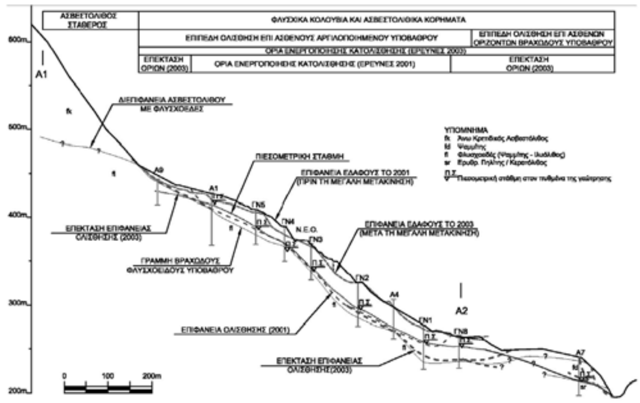


Figure 23. Reactivated Tsakona landslide ([19])

BS 6031:2009, which in Britain is supplementary to Eurocode 7, makes a special note on the influence of brittle and the progressive failure mechanism and the appropriate selection of strength parameters. In paragraph 7.2.4.1 it mentions that one should "first, establish the values of the appropriate ground properties and second, select the characteristic value as a cautious estimate of the value affecting the limit state under consideration taking into account all relevant information".

Vaughan & Walbancke ([42]) have long recognized some important sources of errors and uncertainties in limit equilibrium analyses of long – term first – time slides on stiff clays. The most important are the pore pressures distribution, the effects of fissures, the strength anisotropy, the progressive failure mechanism of brittle clays and the strain rate effect.

Recently, Chowdhury ([16]) presented an iterative procedure on limit equilibrium analyses, which takes into account the softening/weakening of the soil as it happens during the progressive failure.

5.1. Design with Eurocode

The currently valid design standard for European Union countries is the EN 1997-1:2004 Eurocode 7: Geotechnical design - Part 1: General rules. Thirty national members (the 27 EU countries plus three countries of the European Free Trade Association) and 19 affiliates (non EU countries) are part of the European Committee for Standardization (CEN: Committé Européen de Normalisation) network.

Concerning the Balkans, Bulgaria, Croatia, Greece and Romania are national members and Albania, Bosnia and Herzegovina, FYR of Macedonia, Montenegro and Serbia are affiliates. Moreover, Turkey is an affiliate. It is the responsibility of the CEN National Members to implement European Standards as national standards and they have to withdraw any conflicting national standards.

Greece, as well as most of the national CEN members, has chosen the design analysis 3 (DA 3) for the overall slope stability calculations of natural or cut slopes, without structural elements. According to this:

The design strength parameters are:

- $c_d' = c_k'/\gamma_M$, $\tan(\varphi_d') = \tan(\varphi_k')/\gamma_M$, $c_{u,d} = c_{u,k}/\gamma_M$

  where the subscript k stands for representative values and subscript d for design values
  γ_M is partial factor of safety.

- For undrained conditions it is γ_M=1.40, while for drained conditions it is γ_M=1.25.

- The partial factors, γ_F, for the actions are: a) $\gamma_F=\gamma_G$=1.0 for permanent loads and b) $\gamma_F=\gamma_Q$=1,30 for unfavorable or 0 for favorable loads for mobile loads.

5.2. DIN 1054

Germany adopted Design Approach 2. According to DIN 1054:2005 ([18]) Corrigendum 4:2008 the influence of the weathering on rock or rock – like soils (e.g. stiff fissured clays) shall be considered by reduction of the shear strength parameters. For overall limit state stability (GZ 1C) (GEO-3) the design strength parameters are:

$$c_d' = c_k'/\gamma_c,\ \tan\varphi_d' = \tan(\varphi_k')/\gamma_\varphi,\ c_{u,d} = c_{u,k}/\gamma_{cu}$$

where γ_c, γ_φ and γ_{cu} are partial factors of safety of strength parameters:

- Load Case 1 (CA1+SC1, corresponds to the "persistent design situation" according to DIN 1055-100:2001-03): $\gamma_c=\gamma_\varphi=\gamma_{cu}$=1.25 for strength parameters, $\gamma_F=\gamma_G$=1.0 for permanent loads and γF=γQ=1,30 for unfavorable variable actions.

- Load Case 2 (CA1+SC2 or CA2+SC1, corresponds to the "transient design situation" according to DIN 1055-100:2001-03): $\gamma_c=\gamma_\varphi=\gamma_{cu}$=1.15 for strength parameters, $\gamma_F=\gamma_G$=1.0 for permanent loads and $\gamma_F=\gamma_Q$=1,20 for unfavorable variable actions.

- Load Case 3 (CA3+SC2 or CA2+SC3, corresponds to the "accidental design situation" according to DIN 1055-100:2001-03): $\gamma_c=\gamma_\varphi=\gamma_{cu}$=1.10 for strength parameters, $\gamma_F=\gamma_G$=1.0 for permanent loads and $\gamma_F=\gamma_Q$=1,00 for unfavorable variable actions.

In paragraph 12.3, DIN 1054:2005 states that in brittle soils the possibility of progressive failure mechanism shall be investigated. Moreover, it suggests that for slopes the possibility of reactivation of geologically predetermined slip surfaces shall be taken into consideration.

5.3. BS 6031 practice for long term stability

BS 6031 ([9]) distinguishes between the peak, the critical state and the residual strength. It proposes a rule of thumb that for plasticity index PI>25% the residual angle of shearing resistance is $\varphi_r \approx 10^o$, adopting a variation similar to that of Bishop *et al* (1971) and Lupini *et al* (1981).

For fine soils with PI>25% and no preexisting relic shear surfaces BS 6031 proposes to use design values based on φ'_{pk}, c'_{pk} in conjunction with the partial factors of safety of BS EN 1997-1:2004 and its National Annex. The critical state strength parameters φ_{cv}', c_{cv}' should be considered when "significant displacements are likely to occur over the design life of the slope". If relic shear surfaces are present BS 6031 suggests using the residual strength parameters as the design values.

Concerning the design approaches, BS 6031 suggests DA1 for overall stability, while DA2 must be also considered if the external loads are expected to be critical. DA2 is usually more critical and it also results in an overall FS=1.25 for long term conditions.

With respect to reactivated slides, when design includes residual strength parameters, BS EN 1997-1:2004, 11.5.1(8) states that partial factors normally used for overall stability need not be appropriate. BS 6031, proposes a partial factor for the residual angle of shearing resistance, φ_r, about γ_M=1.1, provided that c_r=0.

If the soil is brittle the careful consideration of the possibility of progressive failure is urged by BS 6031. The designer is free to choose an appropriate approach.

6. CONCLUSIONS

The brittle behaviour of stiff natural clays manifests the overall behaviour of natural and cut slopes. Plasticity and clay fraction influence the brittleness, which greatly affect the progressive failure mechanism, and the residual strength, which is important for reactivated slides. Time dependent softening and strain dependent weakening, along with creep phenomena influence the strain softening and the pore pressures. For normal slope design Eurocode 7 and national application documents propose partial factors γ_M.

For first – time slides, the progressive failure mechanism of cut slopes in stiff and fissured clays results in an average operational strength along the slip surface lower than the peak strength (c_{ps}', φ_{ps}') due to soil's softening and weakening. Failure may take some decades to occur and this is attributed mainly to the gradual pore pressure build up towards the steady state condition and to the gradual softening of the soil ([15], [37], [42]).

The fully softened strength (c_s', φ_s') concept has been applied for the operational strength of these delayed failures ([35]), which is found to be close to the laboratory post – rupture strength (c_{pr}', φ_{pr}'). Usually a small c_s' value and a φ_s' between φ_{ps}' and $\varphi_{cv}' \approx \varphi_{NC}'$ applies on the actual slip surface, which is not the critical slip surface that results from a calculation with c_s', φ_s'. As slope stability analyses correspond to low confining stresses, values of post – rupture strength cpr'=0 to 10 kPa and φ_{pr}' slightly higher than φ_{cv}' or $\varphi_{pr}' = \varphi_{cv}' \approx \varphi_{NC}'$, could be adopted. Moreover, it is reported ([12]) that the post-rupture strength is more pronounced at low to moderate stress. For design of slopes within brittle soils BS 6031 urges for the careful consideration of the possibility of progressive failure, while DIN 1054 states that the possibility of progressive failure mechanism shall be investigated. The designer is free to choose the appropriate method, while no reference is made for the required partial factors of safety.

Concerning reactivated slides, the residual strength parameters (c_r', φ_r') apply on the slip surface. A small cohesion lower than 5kPa may be taken into account, which can be justified by the shear creep and the non – linear nature of φ_r'. The shear creep may induce a progressive type of failure. Moreover, it has been reported that the stability of reactivated landslides is very sensitive to small variations of the phreatic surface ([19]). The pore pressure accumulation due to creep could therefore become critical on some occasions. For reactivated landslides, when design includes residual strength parameters, BS 6031 states that partial factors normally used for overall stability (as the ones of paragraph 5.1) need not be appropriate and proposes a partial factor for the residual angle of shearing resistance, φ_r, about γ_M=1.1, provided that c_r=0.

NOTATION

ε_q deviatoric strain (e.g. $\varepsilon_q = 2(\varepsilon_v - \varepsilon_a)/3$ in triaxial device)

ε_v volumetric strain (e.g. $\varepsilon_v=(\varepsilon_v+2\varepsilon_a)$ in triaxial device)

σ_n normal stress

τ shear stress

φ_μ angle of interparticle friction

φ_{cv} critical state strength angle of shearing resistance

φ_{max} maximum frictional resistance shearing resistance

CF clay fraction

c_d, φ_d design strength parameters: cohesion and angle of shearing resistance

c_k, φ_k characteristic strength parameters: cohesion and angle of shearing resistance

c_{mob}, φ_{mob} mobilized or operational strength parameters: cohesion and angle of shearing resistance

c_{pr}, φ_{pr} post rupture strength parameters: cohesion and angle of shearing resistance

c_{ps}, φ_{ps} peak strength parameters: cohesion and angle of shearing resistance during slip formation

c_r, φ_r residual strength parameters: cohesion and angle of shearing resistance

c_s, φ_s fully softened strength parameters: cohesion and angle of shearing resistance

e voids' ratio

FS factor of safety

LL liquid limit

M critical slope

NC normally consolidated

PI plasticity indes

p mean effective stress (e.g. $p=(\sigma_v+2\sigma_a)/3$ in triaxial device)

q deviatoric stress (e.g. $q=\sigma_v-\sigma_a$ in triaxial device)

SCL sedimentation compression line

v specific volume ($v=1+e$)

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Exposed Aggregate Concrete Pavement Design, Construction, and Functional Performance: A comparison of European and US experiences

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ABSTRACT

In 2006, a group of highway agency and paving industry representatives from the USA visited Europe in search of current practices for the design, construction, maintenance, and rehabilitation of concrete pavements. In every country visited, examples of exposed aggregate concrete were demonstrated. While not a new discovery, the discussions did invoke a renewed interest in this technology. In 2007, a follow-up tour of Europe was conducted, where texture and noise measurements were conducted using the same technologies as those commonly used in the USA for these purposes. From this, the functional performance of exposed aggregate pavements was compared to that of surface textures commonly constructed in the USA.

More recently, a large collaborative effort has been initiated by the U.S. Department of Transportation Federal Highway Administration, the National Concrete Pavement Technology Center, and numerous other partners both in the USA and in Europe. The objective is to conduct field demonstrations of exposed aggregate and two-lift concrete pavement technologies in the USA. In 2008, the first large scale demonstration project under this initiative was constructed in the State of Kansas.

From these various activities, it can be concluded that exposed aggregate concrete pavement surfaces are viable for use in the USA under certain circumstances. Furthermore, there is the possibility of applying lessons learned in the USA on projects in Europe. The net result is a global technology exchange where both partners can benefit.

KEY WORDS : exposed aggregate, two lift, noise, texture, friction, functional performance

1. INTRODUCTION

1.1. Functional Demands of our Roadways

When most pavement engineers think of performance, chances are they will default to how a pavement fails in terms of cracking or faulting. When viewed over time, these types of observations are an indication of the structural performance of a pavement.

If the same question is asked of the public, however, they will often cite different things. They will mention how rough a road is, or possibly how safe they "feel" in a rainstorm, or even how loud a pavement is. These indicators, in part, define the functional performance.

Generically, the performance of a pavement can be described by its long-term response to imposed loads and environmental influences. However, there is a distinction between structural and functional performance. The former describes the load carrying capacity of the pavement while the latter is an indicator of how the pavement can affect the human experience or address societal demands.

Functional performance includes things such as smoothness, friction, noise, splash & spray, surface drainage, and rolling resistance. Other characteristics include tire wear, vehicle wear, and reflectivity & illuminance. Affecting these surface characteristics are numerous pavement properties, with the most important being surface texture. These "bumps and dips" in the road range in size from long rolling undulations to asperities that cannot be seen with the naked eye. Other important pavement properties include the degree of permeability and porosity, cross-slope, and mechanical impedance (stiffness). Even the color of the surface will also affect some surface characteristics both directly and indirectly.

1.2. European-US Collaboration

In 2006, a group of highway agency and paving industry representatives from the USA visited Europe in search of current practices for the design, construction, maintenance, and rehabilitation of concrete pavements (Hall et al., 2007). In every country visited, the importance of functional performance was stressed, and in response to these needs, examples of exposed aggregate concrete (EAC) pavements were commonly demonstrated. While not a new "discovery", the discussions did invoke a renewed interest in this technology, citing similar findings from a similar tour in 1992 (Darter et al., 1992).

As a result of this trip, the final report recommended further evaluation of this technology in USA (Hall et al., 2007). In 2007, this began with a follow-up tour of Europe where texture and noise measurements were conducted using the same technologies as those commonly used in the USA for these purposes. During this tour, there was collaboration with many of the same countries that were visited the year prior; namely, Austria, Belgium, the Netherlands, and Germany.

At the same time, planning began for construction of a two-lift concrete paving project in the State of Kansas that would employ – in part – an EAC surface. As part of this effort, additional connections were made with concrete pavement experts in the USA and Europe, and the specific design, materials, and construction techniques were "translated" (CP Tech Center, 2010). Not surprisingly, this proved difficult in some instances due to differences in the availability and evaluation of materials between the two continents. Still, the project in Kansas was successfully constructed in 2008.

1.3. Definition of Exposed Aggregate Concrete

Exposed aggregate concrete is a technique for surfacing/texturing concrete pavements where the mortar fraction in the vicinity of the surface is intentionally removed to leave the larger aggregates exposed, and thus to serve as the contact surface for traffic.

While not necessary, EAC surfaces are commonly constructed using a two-lift "wet on wet" paving process. The top layer thickness typically ranges from 38 to 70 mm (Hoerner and Smith, 2002), and the mix contains fine siliceous sand and – more importantly – a high-quality coarse aggregate with an maximum size typically ranging from 4 to 12 mm (Buys, 2004). Because these high quality materials come at a premium (high cost), the use of a two-lift paving process becomes economically viable. Aggregates used in the lower layer of the pavement can be of lesser quality (e.g., lower abrasion resistance), and commonly include recycled materials that help reduce the overall cost of the concrete.

1.4. Why EAC are a Viable Solution

When designed and constructed properly, EAC surfaces results in a hard and durable stone wearing surface. When sized properly as well, the resulting surface can meet or exceed a number of critical functional performance criteria including noise, friction, and splash & spray, among others. Reinforcing this is decades of successful use of this surfacing, with ample field observations to adjust the materials and process in order to approach a more optimized method.

2. USE OF EAC

2.1. Construction Methods

The EAC surface is commonly constructed by applying a set-retarding agent to the newly placed concrete pavement. After some time has passed (typically up to 24 hours), the surface mortar is brushed away from the top of the pavement, exposing a surface of durable aggregates. If brushed at the proper time, no water will be needed during brushing, and excess dust will not be generated.

When designed and constructed correctly, EAC pavements will result in a uniform surface of aggregate particles that are closely spaced and embedded to a depth sufficient for their retention during subsequent trafficking. The characteristics of such a surface often result in lower noise, improved friction, and durability equal to that of conventional concrete pavement texture (Sandberg and Ejsmont, 2002; Hoerner and Smith, 2002; van Keulen and van Leest, 2004; Rens et al., 2004).

To be successful, however, the EAC requires a high-quality concrete. A maximum water-tocement ratio of 0.38 has been cited, along with a minimum cement content of 450 kg/m³ (Sandberg and Ejsmont, 2002). A plasticizer and air entrainer are specified in order to achieve workability and durability. Polish-resistant aggregates should be used with polished stone values (PSV) over 50 (Sulten, 2004; Teuns et al. 2004; Fults et al., 2004).

2.2. EAC in Europe

The use of EAC surfaces in Europe is widespread. Arguably, the most experience with these surfaces is found in Austria and Belgium, however, projects can be found in numerous other countries. Exposed aggregate concrete has also been used in other parts of the world including, but not limited to, Australia, Canada, and the USA.

The Dutch province of Noord-Brabant conducted a study to determine the surface characteristics of EAC pavements (Teuns et al., 2004). Various aggregates, texture depths, curing compounds, and concrete finishing techniques were used in the study to determine the combinations that provided optimal performance. Two Dutch aggregates, a Dutch stone and Grey Quartzite, were used in the study; the Grey Quartzite possessed a higher polished stone value than the Dutch stone aggregate. Several texture depths were evaluated, with the standard depth considered to be one-quarter of the maximum aggregate size. Different retarding agents were evaluated, including lemon and other acid solutions, along with various combinations of retarding agents and curing compounds. One- and two-layer paving systems, as well as a super smoother (finisher), were also evaluated in the study.

Several key measurements and observations were made after construction. For example, it was concluded that texture depth is increased when a super smoother is used with a maximum texture depth of 1.8 mm when used, compared to 1.1 and 1.6 mm when not. The selection of the retarding agent did not appear to make a difference on the results, but it was concluded that lower noise levels were measured when smaller maximum aggregates were used.

In a different test conducted by the Swedish National Road Administration, several concrete and asphalt pavements were tested for abrasion resistance, friction, and noise under heavy traffic (Hultqvist and Carlsson, 2004). The test sections, comprised of both jointed plain and continuously reinforced concrete pavement, were constructed with EAC surfaces to minimize noise levels. Two different maximum aggregate sizes were used in the design of the concrete pavements, 8 and 16mm. Noise was measured using the close proximity (CPX) method.

In comparison to the asphalt pavements constructed on the same job, initial tests revealed that the EAC pavements with 16-mm and 8-mm stones provided noise levels that were 1.0 to 1.5 dBA and 3.0 to 3.5 dBA lower, respectively (Hultqvist and Carlsson, 2004). The noise emissions of the 16-mm EAC and HMA sections were found to be identical after one year. However, the 8-mm EAC section actually produced quieter noise levels after one year. Four years after construction, the noise levels from all of the pavements had increased an additional 0.5 to 1.2 dBA.

When EAC surfaces are used in Austria, a combination surface retarder and curing compound is commonly applied immediately after placement. The uniformity of application is critical to achieving an optimum surface. The automated equipment used for this purpose is illustrated in Figure 1 (left). In the Netherlands and Belgium, a technique employing a plastic film applied to the fresh concrete is common. This is shown in Figure 1 (right).

Figure 1 – Applying surface retarder and curing compound (left) or plastic film (right)

Sawcutting operations for EAC proceed using the same accepted practices as for conventional concrete paving. The degree of set and anticipated weather are key considerations in this timing. This is shown in Figure 2. The brushing operation that is most often used to remove the surface mortar is also a function of the degree of set of the concrete. Timing of this operation is critical, and is often determined on test strips well in advance of the paving. Variables that affect the timing of strength gain (e.g., cold weather) will also affect the ideal time for brushing, so measures are taken to gauge the sensitivity of these variables to the timing of the brushing. A close-up of the brushing technique that is commonly used is shown in Figure 3 (note that a power brush can also be seen in the background of Figure 2).

Figure 2 – Sawcutting operations followed by brushing for surface mortar removal

The appearance of the final EAC surface will depend on the type of coarse aggregate used, the size of the aggregate, and the timing and thus depth of mortar removal. Typical EAC surfaces photographed from pavements in Belgium and Austria are shown in Figure 4 and Figure 5.

Figure 3 – Close up of brushing for surface mortar removal

Figure 4 – EAC pavement in Belgium

Figure 5 – EAC pavement in Austria

2.3. EAC in North America

Although EAC pavements are commonly used in European countries, the technique has not been routinely used in the United States. Two such projects have recently been reported in Quebec (Thebeau, 2004).

Until recently, the only large-scale EAC pavement project in the United States was completed in 1993 on Interstate 75 in downtown Detroit, Michigan (Hoerner and Smith 2002; Kuemmel et al. 2000; Weinfurter et al., 1994; Smiley 1995; Smiley 1996; Smith 2001). This project aimed to demonstrate the effectiveness of the EAC paving concept along with other technologies identified during the 1992 European Scanning Tour on Concrete Pavements (Darter et al., 1992).

The EAC pavement was comprised of a 250-mm jointed concrete pavement constructed in two lifts. The top layer of the pavement was 65 mm thick with polish-resistant aggregates, and the bottom layer was 190 mm thick with conventional aggregates (Kuemmel et al. 2000; Weinfurter et al., 1994). The lifts were bound using a "wet-on-wet" procedure. A conventional jointed reinforced concrete pavement was constructed nearby as a basis for comparison, textured with transverse tines spaced 25 mm apart (Kuemmel et al. 2000; Weinfurter et al., 1994).

Both sections were tested for friction and tire-pavement noise levels. While there was not much difference in friction levels between tests conducted one year and five years after construction, the EAC pavement did not measure well for noise (Kuemmel et al. 2000; Smiley 1995; Smiley 1996). The section provided a reduction of only 0.4 dBA in exterior noise levels.

Researchers believe that the disappointing values may have resulted from too much macrotexture on the EAC surface, combined with excessive spacing between the coarse aggregate particles. This excessive spacing was a result of large sand particles. Researchers subsequently advised that sand particles larger than 1 mm should be eliminated from the top layer of concrete (Smith 2001).

3. NOISE INTENSITY TESTING IN EUROPE (NITE) II

In 2004, Dr. Paul Donavan of Illingworth & Rodkin was asked by both the California Department of Transportation (Caltrans) and FHWA to conduct a survey of European pavements using a tirepavement noise measurement technique termed On-Board Sound Intensity (OBSI) (Darter, 1992;

Hall et al., 2007). The test program was termed Noise Intensity Testing in Europe (NITE), and included measurements on over 60 unique test surfaces. Dr. Donavan reported that while there were measurements on a number of exceptionally quiet surfaces, many of the surfaces had levels that are comparable to similar pavements found in the USA (Darter, 1992).

In a parallel effort to help address the noise issue, Iowa State University in 2003 initiated what is now the National Concrete Pavement Technology Center, facilitating a broad industry coalition formally designated as the Concrete Pavement Surface Characteristics (CPSC) Program (Cackler et al., 2004). This partnership includes the FHWA, American Concrete Pavement Association (ACPA), and now various State Departments of Transportation (DOTs) through part of a pooled fund study. Numerous other private and public sector partners are also involved in varying capacities. To date, the major work element of this program has been a comprehensive field experiment (Ferragut et al., 2007). Included has been testing of over 1500 unique test sections representing approximately 500 nominal textures, each evaluated for noise, texture, friction, and other relevant measures.

One of the goals of the CPSCP is to, "develop and evaluate innovative construction techniques that have the potential to significantly reduce noise." (CP Tech Center, 2005) To address this, the work conducted by Dr. Donavan was built upon under a program termed NITE II where a more targeted evaluation of European quieter pavement technology was conducted.

The NITE II testing was conducted in 2007 and included testing in Austria, Belgium, the Netherlands, and Germany. Visits were made to each of these countries during the 2006 concrete pavement scan tour, and as such recent contacts were made with the local transportation authorities as well as the concrete industry (Hall et al., 2007).

Most of the concrete pavements selected were textured with EAC. In the countries that were visited, this is the most commonly used technique for concrete pavement texturing, and has been reported by some as "whisper concrete", which at the very least should underscore its perception when compared to older techniques for concrete pavement surfacing (Sandberg and Ejsmont, 2002).

As part of NITE II, 18 sites were visited with six in Austria, three each in Belgium and the Netherlands, and six in Germany. Since many of the sites had multiple test sections (unique nominal surfaces), a total of 68 test sections were evaluated totaling nearly 20 km in length. Sixteen of the test sections were EAC, with 6 in Austria, 3 in Belgium, 1 in the Netherlands, and 6 in Germany.

3.1. Measurement Scope and Methods

Testing for OBSI adhered to the specification AASHTO TP 76-08 (AASHTO, 2008). As illustrated in Figure 6, the test vehicle used for the testing was an Opel Zafira. Except where safety did not permit it, all testing was conducted while operating at 97 km/hr (60 mph).

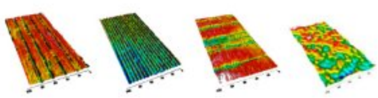
Figure 6 – OBSI bracket and test vehicle (Opel Zafira)

In addition to OBSI, texture testing was conducted using RoboTex, an innovative threedimensional texture measurement system developed in 2005 (Ferragut et al., 2007). Illustrated in Figure 7, RoboTex was developed as a device to establish a better balance between data resolution and efficiency in data collection. It is built around a LMI-Selcom RoLine line laser sensor, and fixed atop a remote-controlled robotic chassis. As a texture profiler, it meets Class CE device requirements per ISO 13473-3 (ISO, 2002).

Figure 7 – RoboTex three-dimensional pavement texture measurement system

Since RoboTex evaluates texture in three dimensions, calculations are made in both the longitudinal and transverse directions. Differences in the texture in both directions are readily apparent for conventional US textures such as tining and grinding (see Figure 8). Surfaces such as EAC are more isotropic, which means that the texture characteristics in both directions are very similar. Subtle differences are still important, however, and can often be related to specific methods in construction/texturing (Rasmussen, 2008).

Figure 8 – RoboTex texture profile samples for longitudinal tining, diamond grinding, transverse tining, and exposed aggregate



3.2. Measurement Results in Europe

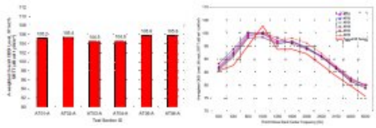
In October 2007, the testing began in Austria, proceeded to Belgium, to the Netherlands, and concluded in Germany. The project sponsors and others involved in the study were constantly updated via an Internet "blog" site at www.surfacecharacteristics.com. This is portal provided quick releases of data that allowed for suggestions to be made in time to potentially incorporate them into the test program. In the following sections, a summary of the key observations at each of the countries is made.

3.3 Austria

Six unique EAC surfaces were evaluated in Austria. One was on A2 just south of Vienna which was a location that was visited by the 2006 scan tour, and where subjective observations were made of the relatively quiet pavement. The remaining sections are located at various points along the A1 highway, which travels the length of Austria, linking Vienna and Salzburg. Maximum aggregate sizes of 8 and 11 mm have both been used routinely, along with other subtleties in the mix design. All of the surfaces were between 4 and 15 years old when evaluated, and as principal motorways in Austria, have been subject to very heavy traffic.

As Figure 9 illustrates, testing of the six surfaces revealed OBSI levels that were all approximately the same, ranging from 104.5 to 105.8 dBA (ref 1 pW/m²). The spectral content of the OBSI levels is interesting, however, as it is well distributed across three or four third-octave bands. This is compared to many pavements in the US that have characteristic peaks at 800 or 1000 Hz, with much lower levels at other bands (Ferragut et al., 2007). To illustrate this, typical spectra for a transversely tined pavement in the US with 25 mm spacing is shown. From a psychoacoustics perspective, the more broadband nature of this source could account for the more "pleasing" nature of these pavements compared pavements measured to have more tonal content (Zwicker and Fastl, 1999).

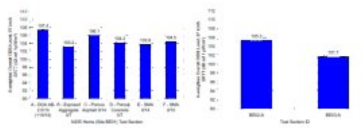
Figure 9 – OBSI overall levels and spectral content for Austrian test sections



3.4. Belgium

In Belgium, the first EAC that was evaluated was along a test site including a series of pavement sections constructed in 1995 (Flemish, 1997). The objective was to evaluate the effectiveness of two quiet concrete pavement alternatives: a "fine" EAC surface consisting of 7 mm maximum aggregate size, and a porous concrete. These were constructed end-to-end with four asphalt surfaces including a dense-graded HMA, porous asphalt, and two SMA's. The results of the measured OBSI levels for each of these sections can be found in Figure 10. As expected, there have been significant changes to the levels – on a relative basis – since the early measurements collected by the sponsors (Flemish, 1997).

Figure 10 – Overall OBSI levels for Belgium test sections



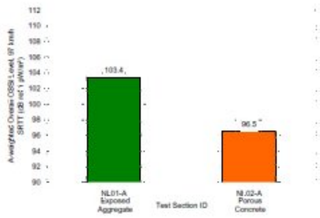
To the right of this figure are the results of two very different EAC surfaces found on the N49 (E34). The BE02 section consisted of a 20 mm maximum aggregate size, while the BE03 was constructed with 6 mm. Photographs of these two sections can be seen in Figure 11. The finer texture of the 6 mm aggregate is the likely contributor to the lower noise level. Assuming that the durability or friction of this surface is not compromised, this is a promising surface for EAC as it has a much lower level than other EAC measured throughout Europe.

Figure 11 – Photographs of the BE-02A (left) and BE-03A (right) test sections

3.5. The Netherlands

In the Netherlands, the first two test sites measured consisted of one nominal surface each. The EAC surface revealed a level that was within the range of those measured in Belgium. Figure 12 shows this level compared to a porous concrete surface constructed on a pavement system called Modieslab, developed as a result of the Dutch Roads to the Future program, and reported on by the recent scan tours (Hall et al., 2007; Fults, 2005). The IPG test track near Kloosterzande was also evaluated while in the Netherlands, however the results are not given herein.

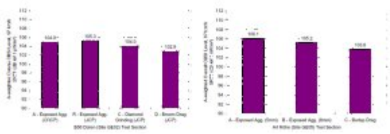
Figure 12 – Overall OBSI levels of two test sections in the Netherlands



3.6. Germany

Eleven surfaces at five locations were evaluated in Germany. Two of these locations had additional opportunities to evaluate different nominal pavement surfaces end-to-end. On the B56 near Düren, four concrete pavement textures were evaluated that were also evaluated under NITE I. The resulting OBSI levels are shown in Figure 13.

Figure 13 – Overall OBSI levels for two German test sites



On the test section along the A4 near Röhe, the two most commonly used texturing techniques for German concrete pavements are found – EAC (which is increasing in use), and burlap drag (until recently, the standard texture). As shown to the right of Figure 13, the OBSI level measured on the drag surface is found to be 1.4 to 2.3 dB lower than that on the EAC. It is possible, however, that the EAC found on this section is higher than typical for German practice, since the other sections measured on the GE-01 test site measured at 102.9 dBA (ref 1 pW/m²).

4. RECENT FIELD TRIAL OF EAC IN KANSAS (USA)

In 2008, a project was constructed in the State of Kansas, in Saline County, where two-lift concrete pavement construction was used in lieu of conventional single lift construction (CP Tech Center, 2010). As part of this project, a premium coarse aggregate was used in the surface course, and EAC used for approximately 1 km of surface, 2 lanes wide. The project was constructed along a major rural highway, Interstate 70. However, to help mitigate some of the risk involved, a test section was first constructed on a small road in another part of the state.

4.1. Design and Materials

The State of Kansas does not contain any approved sources of aggregate that are suitable for EAC surfaces. As such, rhyolite aggregate had to be imported from 550 km south in the State of Oklahoma. Because this required transport by truck, the additional cost was significant. Furthermore, while the material was of better quality than that locally available, it still did not meet the stringent requirements commonly cited from European specifications (FSV, 2007). For example, the PSV was 30 to 35, compared to a specified minimum of 50. The other aspects of theconcrete mixture and design were targeted using guidance provided by the Austrians (FSV, 2007).

4.2. Construction

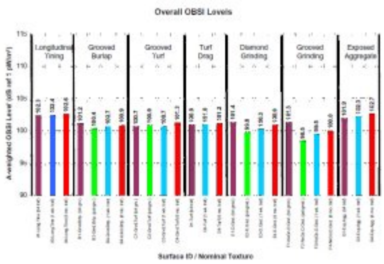
The pavement was constructed with equipment commonly used for conventional paving in the USA. One significant difference was the simultaneous use of two pavers that were responsible for the placement of the bottom and top lifts, respectively. The bottom lift was designed as 300 mm thick, and the surface course was nominally 40 mm. However, during construction, there was significant variation in the thickness of the surface layer. In some cases, the surface course was considerably thicker than nominal – up to 100 mm. This possibly indicates displacement of the underlying concrete as the second lift is being placed.

The EAC was created by applying a surface retarder immediately after placement. Polyethylene sheeting was then applied for the 5 to 6 hours of curing that was typical prior to brushing the concrete. The sheeting was removed in 30 m segments, and the brushing was conducted with two power brooms with wire bristles. After brushing, a wax-based curing compound was applied, and sawing of the joints proceeded at a time that was appropriate to minimize the potential for uncontrolled transverse cracking.

4.3. Surface Characteristics Measurements

Noise, texture, and friction measurements were collected on the various pavement surfaces along the project shortly after construction, and once again after the first winter had passed. Figure 14 illustrates the results of the noise testing, conducted with the OBSI technique. In this figure, the EAC results are shown on the right for three different times, and can be compared to the other textures used on this project including various combinations of drag, longitudinal tining, grooving, and diamond grinding. From this figure, it can be seen that the EAC is similar in level to other conventional textures.

Figure 14 – Overall OBSI levels for Kansas test sections



5. COMPARISONS OF EAC IN EUROPE AND USA

5.1. Design and Materials

From the Kansas project, and as part of discussions with other owner-agencies in the USA, it appears that one of the most difficult challenges in the use of EAC in the USA is the availability of suitable quality aggregates. While the USA does have numerous sources of qualifying material, they are isolated in select areas, and transport of these materials to distant projects becomes economically disadvantageous.

There will also be difficulty in overcoming the institutional inertia present in the US paving industry. Innovation will often only happen when the owner-agency and the contractor are both supportive. Furthermore, initial costs will be an overwhelming motivator to change, which can be difficult to justify in surface texture, considering that the EAC will be significantly more expensive than conventional texturing practices.

5.2. Construction

The placement technique used on the Kansas project was similar to projects that are constructed in Europe. Two pavers were supplied with concrete manufactured at the same batch plant. To minimize confusion upon delivery of the concrete, a simple "colored placard" system was used – at the batch plant, by the truck driver, and at the point that the concrete is placed into the paving train.

One notable difference was the absence of transverse waves that in the European operations are commonly produced by an oscillating transverse beam. The waves are partly filled with mortar by the longitudinal smoother, and are exposed again during the brushing process. This is likely exacerbated by the use of a gap-graded mixture with a considerable mortar fraction that can be separated by these actions.

The most interesting and relevant difference in the typical European projects and that in Kansas is the orientation of the crushed surfaces of the aggregates. As shown in Figure 15, the aggregates were typically aligned with a "flat" face oriented parallel to the pavement surface. It has been reported that an as-constructed aggregate structure that results in a "plateau with gorges" texture will prove quieter than a texture with "peaks and valleys" (Sulten, 2004).

Figure 15 – As-constructed EAC surface on Kansas project

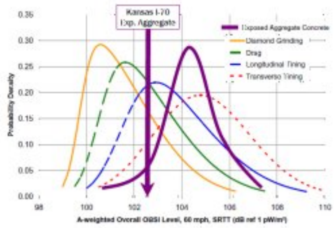
The cause of this is still being debated, but one hypothesis is the use of the smoothers (or lack thereof in the USA). Another is the type of vibrators used in the Kansas project versus those commonly used in Europe.

5.3. Surface Characteristics

There is extensive use of EAC in Austria and elsewhere in Europe. Over the years, many refinements have been made to the surface in order to balance safety (measured by texture depth and friction), durability, cost, and in recent years, noise. Also of importance is the durability of the surface over time.

Figure 16 illustrates a number of concepts. First, the various "bell curves" shown on this plot represent normalized distributions of the various concrete pavement textures commonly used in the USA. The heavier line represents the normalized results of the various EAC sections evaluated in Europe, Canada, and previously in the USA (State of Michigan). The arrow identifies the noise level that was initially measured on the EAC in the State of Kansas, and represents one of the quietest of all EAC surfaces evaluated using OBSI.

Figure 16 – Normalized for various concrete pavement textures including EAC



Studies have shown that the use of smaller aggregates in the concrete at the surface provided better noise reduction levels (Sandberg and Ejsmont 2002; Hultqvist and Carlsson 2004; Teuns et al., 2004; Fults et al., 2004). However, the comparison between the Kansas project and other projects in Europe did not reinforce this observation. Instead, the geometry of the texture previously described is the likely factor that led to differences in the noise levels.

A key consideration, however, is the durability of the pavement texture. As Figure 14 illustrates, there are already some changes in the measured noise levels after one winter maintenance cycle.

Additional changes in the texture and noise are expected in the years to come. That said, projecting the current noise levels of the various European pavements back to a "year zero" condition still reveals that the Kansas pavements are generally quieter. The functional performance of the pavement must be tracked in the coming years in order to verify that the texture is not changing at an exceptionally high rate.

6. CONCLUSIONS

6.1. What can the USA try next?

The 2008 demonstration project in Kansas has spurred interest in this technology in the USA. Since that time, there have been numerous discussions about additional projects in Kansas and other states including Georgia and California, among others.

In the USA, the potential for noise reduction is a commonly cited benefit of EAC surfaces. However, as the data collected on more conventional concrete pavement surfaces has shown, this alone cannot be a reason to employ this technique. An emphasis on the durability of the surface must be considered as well… not only for acoustical durability, but for other functional performance indicators including friction and splash & spray.

As additional projects are tried, additional variants of the technique should also be tried, including more optimization of concrete mixtures and use of combination surface retarder/curing compounds.

6.2. What can Europe try next?

There continues to be interest in the project in Kansas by the international community.

Representatives from various European nations have visited the project since 2008 to make firsthand observations of the as-constructed surfaces. During these visits, the dialog continues about the particulars of the project with the Kansas Department of Transportation, Koss Construction, and Guntert & Zimmermann.

From these discussions, alternatives to some of the construction practices commonly used in Europe are being proposed. For example, the paving equipment used on the Kansas project did not employ an oscillating transverse beam or a longitudinal smoother, and thus had the potential for fewer disruptions and more uniformity in the paving concrete. Furthermore, the specific vibrators used in Kansas were also identified as a potential factor contributing to the resulting surface.

As these and other potential differences are identified, it is possible that further refinement and optimization can be made of the decades-old practice of using EAC surfaces in Europe.

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Don’t take risks: Consider Mining Geotechnics

Tony Meyers, Vice President (Australasia), International Society for Rock Mechanics and Principal, Rocktest Consulting, Australia

This century has seen, and will continue to see, significant changes in the size and operation of pits throughout the world. Contributing to these changes has been the increasing demand for minerals as populations increase. Over the same period, there has been a significant reduction in the rate at which many mineral resources are being discovered, especially at shallow depths. When they are discovered, alternate land use requirements and environmental pressures often made it difficult to convert the resources into reserves. These issues have been fundamental drivers for existing and proposed pits going deeper. New technologies and techniques can make doing so cost effective. However, going deeper can introduce new geotechnical issues for operators.

GroundProbe SSR (Slope Stability Radar)

Historically, these issues may only have been considered when a pit was initially designed and/or if pit wall instability occurred. However, in the past decade, increasingly regulated safety requirements have been forcing operators to take an ongoing riskbased approach to pit wall design and management. In addition, financial pressures have strengthened the link between the economics of a mining operation and the geotechnical risks associated with it. This article considers some of the geotechnical issues an operator must consider to address these challenges.

The mass of rock within which every pit slope is excavated is a complex system involving many characteristics that vary significantly throughout the mass. These characteristics include rock strengths and deformability, groundwater pressures and the locations, orientations, spacings and sizes of joints, faults, shears and foliation. However, we can never know what these characteristics are at any location as we cannot test or even see every rock behind a slope.

Changes in many of these characteristics influence other characteristics. For example, vibrations from blasting or natural weathering can open cracks in benches, which can allow groundwater to permeate into the mass, which can produce pressures within the cracks, which can reduce the strengths of the cracks, which can cause the cracks to open, which can allow more water into the cracks. This process, referred to as coupled behaviour, can repeat itself until an instability develops.

The combination of these characteristics makes pit slopes conducive to unpredictable behaviour. As a site’s slopes increase in height, this unpredictability presents increasing challenges for the operators of the site as the frequency of slope instability will often increase.

These challenges are compounded by financial pressures on operators: The pressure to increase batter angles and/or reduce berm widths can result in a reduction in pit wall stability and an increase in the number of instabilities that impact work zones; the pressure to increase output can result in an increase in the number of personnel within a pit as the number of areas being worked simultaneously increases. Both actions increase the likelihood that personnel will be present when instability occurs.

The only way to address these challenges is to have in place a sound, comprehensive pit slope management programme. At the heart of such programme is the concept of geotechnical risk.

The four components of this risk for loss of life (i.e. annual probability of death) can be considered to be:

- The annual probability of a hazardous event occurring from a pit slope (e.g. a rockfall);
- The probability that a person will be within the trajectory of the hazard;
- The probability that a person will be present at the exact time the event occurs; and
- The vulnerability of the person to the impact (ie. the likelihood that the impact will cause death).

The vulnerability component of risk assumes that a person is impacted. To reduce the likelihood for this impact to occur, the quarry operator must reduce sufficiently one or all of the first three components that contribute to the risk. They must be able to demonstrate that the resulting risk is as low as reasonably achievable.

Some of the requirements for operators to implement to enable them to do so follow.

Even small pit walls should be designed by a qualified geotechnical engineer. The designs must be based on a thorough investigation into the engineering characteristics of the intact rocks and rock mass, static and dynamic forces to be applied to the slopes, long-term operational requirements for the slopes, infrastructure requirements (e.g. ramps, haul roads) and overland flows of water and groundwater characteristics.

Antamina Mine, Peru

All production and final batters must be scaled thoroughly. Primary scaling begins as soon as the loading excavator reaches a blasted muckpile. It continues while the excavator waits for trucks. Secondary scaling is carried out after the muckpile has been removed and before the final cleanup of the berm. A dedicated secondary scaling excavator may be used which frees the loading excavator to be used elsewhere. The scaling excavator is often fitted with a long reach boom and a small, toothed bucket. Alternatively it may be configured as a backhoe and fitted with a ripper tyne to enable individual loose rocks to be removed. Secondary scaling can be done effectively by a dozer, located on the berm above, 'chaining' the batters with a heavy link chain, such as a ship's anchor chain.

Overland flows of water towards the crest should be diverted to contour lows with diversion ditches or bunds. Water must not pool on berms, ramps or haul roads; surface and/or subsurface drains being used to channel water off them in a way that will not cause erosion of crests and batters. Water pumped out of a pit must be channelled into contour lows, clay lined raw water dams or a tailings dam. It must not be discharged anywhere near the pit to prevent it permeating back into the pit walls.

BLASTING DAMAGE

In the last decade, economics has driven the increased use of highenergy mass blasts. The energy resulting from these blasts has the potential to damage benches. In an effort to reduce damage and achieve high-quality batters, controlled blasting techniques have become more widespread, primarily for limit blasts but also for some production blasts. These techniques may involve buffer (cushion), trim (postsplitting) and pre-splitting blasting. The availability of electronic detonators has significantly increased the ability of operators to achieve the desired results.

Pre-splitting – once used only on final batters now being used for rockfall control on production batters adjacent to haulage routes

Berms are the primary component in a slope design for limiting the trajectory of falling rocks. As a rule of thumb, final and production berms should have minimum widths of one third and two thirds the height of benches respectively. Additional width (i.e. one to two metres) should be specified to allow for overbreak. Drainage on berms must be maintained, damage repaired and rockfall debris removed.

Bunds constructed on production berms can reduce, temporarily, the risk to persons who must spend time in front of production batters, e.g. blast-hole drillers and shotfirers. Bunds are normally constructed from blasted rock and are at least two metres high. They should be placed at least one third of the height of a bench out from the toe of the batter.

Over the past decade, the use of proprietary flexible rockfall barriers has been steadily increasing in pits. Barriers are generally placed on the berms of final slopes or out from their toes on the pit floor. The cost to supply and install a barrier can be significant. A barrier that has too high or too low an energy capacity or a non-efficient height can be a waste of money. Too low of either can provide personnel with a belief that the risk is less than it actually is. Correct barriers selection should be done by an independent engineer.

Draped rockfall mesh can reduce the risk associated with rockfalls from final batters by controlling the horizontal displacement of rocks, thereby preventing them from launching out from the batter. Mesh is generally suitable for controlling individual rocks up to 0.6m diameter or ravelling masses of small rocks up to ten cubic metres in volume.

Establish exclusion zones to remove persons from areas generating falls of rocks (e.g. adjacent to production digging and below cutbacks) and areas that are potentially unstable. If zones are to be effective they must be rigorously enforced with persons breaching restriction being reprimanded. The zone levels which would normally be created are 'No entry', 'Entry by heavy vehicle with suitable FOP systems', 'Entry by light vehicle – no foot traffic' and 'Entry by foot traffic allowed'.

Benches above all active mining areas should be inspected daily by a trained person and visually monitored throughout the mining cycle. Features of interest include fresh cracks, lowering of the ground surface; bulging on a batter or a berm, water running over a crest or entering cracks, pooling of water, water issuing from a batter, and rocks on berms that have detached from batters.

Any cracks or surface movement on a berm or behind a crest should be monitored with a surface extensometer. Proprietary devices having digital readout and data logging capability are readily available. However, units with manual readout can be just as effective and reliable, and can be used regularly as they are of low cost. All systems can be connected to a movement-activated limit switch which is connected by electrical cable to a remote flashing xenon beacon and 120dB pulsed tone siren powered by a solar-charged 12V battery.

Prism-less laser scanners enable slopes to be scanned from distances up to six kilometres with an accuracy of 25mm to 50mm. As they require no prisms to be mounted on a slope, there are not the safety and time issues associated with installing prisms. There is also not the significant limitation that only those locations where prisms are installed can be monitored, as instabilities seem to occur between prisms. The scanners are often fixed permanently within a rigid steel enclosure located behind the crest of a wall opposite the wall to be monitored.

PHOTOGRAMMETRY

Photogrammetry has become a popular, relatively low-cost, technique for monitoring mining and instability induced changes in slopes over time. A standard digital camera is set up on a levelled tripod and a minimum of two overlapping digital photos are taken of a slope. The photographs are uploaded into proprietary software which creates a 3D photograph and 3D point cloud from the images. The results are used to determine quickly and safely the dimensional characteristics of the slopes and of large structures daylighting out of them.

In-pit radar has been one of the most significant developments in the past 30 years for reducing the risk associated with slope instabilities. Radars enable 270º horizontal and 100º vertical scanning of slopes in real time from a distance of up to 2.8 kilometres without the need for reflectors. Units can have 0.1mm accuracy depending on range and can operate in fog and darkness. Any movement is displayed as colours on a computer screen. Flashing beacons, sirens and SMS warnings can be activated if movements exceed a pre-set amount.

GROUND CONTROL MANAGEMENT PLAN

All sites require a Ground Control Management Plan to formalise all geotechnical systems and processes. The plan provides a systematic approach to the planning, design, management and review of all aspects of work associated with ground control at the site. A plan is useless if it is not going to be used and updated regularly. To facilitate these actions, the plan should be structured in way agreed on by site personnel, company executives and regulators as being the most useful possible for the site. The plan must be written in an easy to read style so that non-technical personnel and executives don't find it too difficult or cumbersome to understand. Readability ensures that it becomes a source of practical information that personnel will refer to for all geotechnical related issues. It must, however, be sufficiently thorough to enable new technical personnel (engineers, managers, geologists, surveyors, technicians etc.) or geotechnical engineering consultants to read through the document and gain an understanding of all geotechnical issues at the site and the procedures used for managing them.

These are just a snapshot of some of the techniques being used in quarries and mines to reduce the risk to personnel. Operators must look at the components of their own risk profile and determine which of these, or the many other available, techniques is applicable to their site.

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(QUARRIES & MINES 2012, pp. 27-31)

The Furnas Type II Stress Measurement Cell

Ludger Suarez-Burgoa

ISRM Young Members' Presidential Group for the South American Region

Since around 1930, methods and tools where developed in the world in order to attain rock mass in-situ stress field measurements (e.g. surface overcoring, flat jack, borehole overcoring, and hydraulic fracturing). Nowadays, there exist diverse methods and equipments that attempt to assess in the best way the natural stress tensor in rock mass. Variations exist from one country to other, each of one tried to develop their own technology, own known-how and own state-of-the art.

This was not an exception in South America. Since the middle of the nineties of the past century, the research group of Doctor João Luiz Armelin, they dependent by the Brazilian State Company FURNAS (today named ELECTROBRAS - FURNAS), and in academic collaboration with some universities of the region, developed the Furnas Type II Stress Measurement Cell, a cell that adopted the overcoring concept to measure in a point the *in-situ* stress tensor in the rock mass (Figure 1).

Figure 1. Doctor Armelin with the Furnas Type II Stress-Measurement Cell

The beginning

The idea of developing the cell begun in the seventies of last century, when Doctor Armelin, still a young graduate geological engineer, worked in the Energetic Company of the São Paulo State (CESP: *Companhia Energética de São Paulo*) for the *Ilha Solteria* Project, in the state of São Paulo (Brazil). There, he have close contact with Professor Manuel Rocha of the National Laboratory of Civil Engineering of Portugal (LNEC: *Laboratório Nacional de Engenharía Civil*) and later he had the opportunity to be close related with the *in-situ* stress tensor assessment campaign of Professor Bezalel Haimson and their collaborators, who were performing hydraulic-fracturing measurement for the *Serra da Mesa* Project (Figure 2).

To this respect, Doctor Armelin commented:

The contact with Professor Manoel Rocha arose still in the CESP. He was a consultant for a variety of works of the CESP in the state of São Paulo. Because we worked in the CESP laboratory, in Ilha Solteria, his presence was frequently in the laboratory and he encouraged several researches about flat jack for the determination of in-situ stresses in sedimentary rock masses… In one opportunity we could talk with Professor Rocha about the triaxial cell and the overcoring techniques.

Figure 2. Professor Bezalel Haimson rounded by a group of Brazilian professional in the hydraulic-fracturing measurements campaign for the *Serra da Mesa* Project in 1975 (Doctor Armelin is at the right hand of Professor Haimson)

Then, Doctor Armelin was part of the professional group designated to arrange the new Rock Mechanics Laboratory of FURNAS, located until now in Goiânia (Brazil). Therefore, he and their comrades decided to develop for this institution an own overcoring cell.

The research

Even though the enthusiasm, desire and ideas about the *own Brazillian cell* was in the minds of these young engineers of the Rock Mechanics Laboratory of FURNAS, the research begun only 20 years after.

It was around the middle of the nineties of last century, when the research project for developing the cell really starts under the support of FURNAS; and for around 15 years all research milestones were accomplished one by one in order to have nowadays (for the beginning of the second decade of this century) the Furnas Type II Stress Measurement Cell.

The research first defined the geometrical dimensions of the future overcoring cell. It looked for the best diameter to length ratio of the core of the cell (where the strain gages are attached), and the proper quantity and position of the active strain gages around the cell core. Other goal in defining the proper geometrical values for the the cell was to obtain a cell to be easy operated in standard boreholes with standard drill-hole tools. These permitted today use the Furnas Type II Stress Measurement Cell with the EW and HW drill series.

After obataining the optimum cell size and desired strain gages arrangement, the next challenge was to reduce the rigidity of the cell core to a minimum possible value. Therefore, some materials were tested in order to finally choose a low viscosity epoxy resign as the cell material.

Then, physical and numerical models, and comparative *in-situ* tests with known international cells (i.e. the CTT and CSIRO cells) were performed, all of these in order to verify the efficience of the developed cell. The physical tests consisted to stress cubes, orthotropic in stress-strain behaviour, to polyaxial normal stresses; and then simulate the complete process of *in-situ* stress measurement with the new cell. This experience was repeated by using 3D numerical finite element method models (Figure 3).

After that, the attention was concentrated to have an efficient and robust data acquisition system module to be

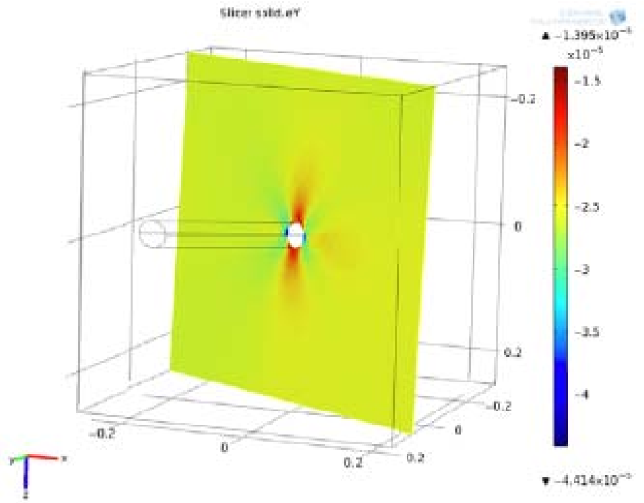

Figure 3. Three dimensional numerical model simulating the stress measurement process with the Furnas Type II Stress Measurement Cell

coupled to the cell (Figure 4), and finally the research finished with the development of a properly software (named CaTMIso) for the data interpretation. This software helps reduce the interpretation time and the possibility to have calculation errors.

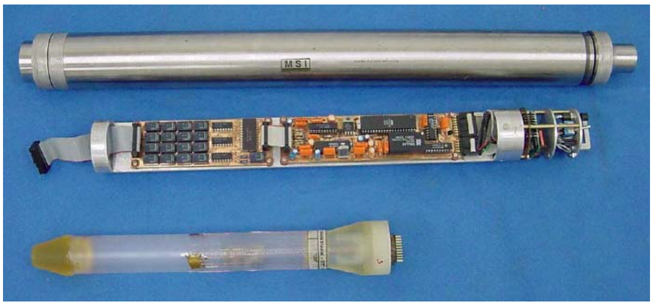

Figure 4. The core cell, the data acquisition system module and the protection tube of the Furnas Type II Stress Measurement Cell

Nowadays the Furnas Type II Stress Measurement Cell is ready to act as a reliable measurement system for punctual stress tensor measurement in the rock mass by the overcoring method. Real tests (i.e. those tests not performed as part of the research program) were just performed in Brazilian projects, which shown to the industry good results.

To this respect, Doctor Armelin commented:

...we performed measurements besides those initials at the Serra da Mesa project, which is the major site of stress assessment present in Brazil… we made also measurements in the Caraíba mine; and we compare the LNEC cell with our cell. Under the interpreters' point of view, which were not us who made the interpretations (referring to FURNAS personnel) because a series of reasons, they adopt a favorable position to the Furnas model cell.

Parallel research

The stress measurements with overcoring methods are still bringing some external influences that can modify the true values; these are mainly the temperature and the rock material mineral or micro fracturing size distribution present in rock mass.

During the development of the Furnas Type II Stress Measurement Cell, parallel research were done with the objective to minimize the temperature effect in the strain measurements of the extensometers and deduce the size effect of rock mass on stress measurements.

Future research

Future research is programmed in order to achieve full temperature compensation during the overcoring process, trough an adequate instrumentation.

To this respect, Doctor Armelin commented:

... the technical difficulty that still remains is in respect to the temperature compensation of the extensometers; this is a suggestion for us in order to maintain the research in this direction.

Technical data

The reference cell, from which this new cell was developed, was basically a foreign model, which for its initial version it proposed seven active strain gages. Because of the name "II" in the actual Furnas cell, one can perceive that an antecessor to the Type II existed, the Furnas Type I Stress Measurement Cell. This cell was primary the Brazilian version of the foreign model, but the Type II added the following improvements:

- It has a low rigidity core cell
- It has more strain gages for measurements in order to have more redundancy in more directions
- Their readings are more sensible
- It is applicable for conventional drilling equipments and tools

The core cell is made of a polymer (i.e. low viscosity epoxi resin), cylindrical in shape with a diameter to height ratio of 1/6 and 1 mm of thickness. It has 12 active strain gages (four strain gages in three groups), these coupled to the external face of the low rigid cylinder and protected externally with a thin layer of epoxy. Each strain gage group is located 120° to each other when looking the transversal section of the core cell, and they are rotated some grades around their own radial axis. Also, each strain gage in a group, is rotated 45° to each other (Figure 5).

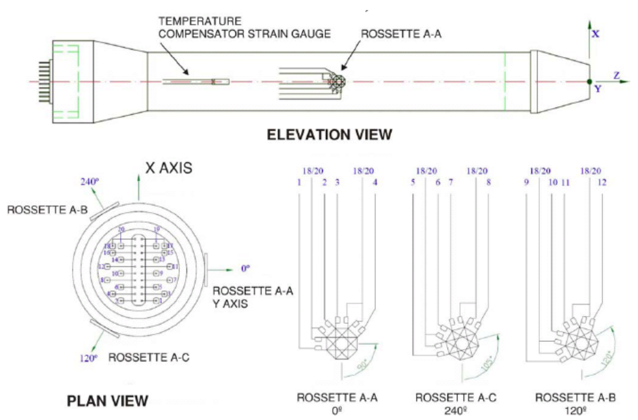

Figura 5. The strain gages arrangement in the core cell of the Furnas Type II Stress Measurement Cell

In the interior of the cylinder, the core cell has one dummy strain gage for compensations, this coupled to a rock material block (i.e. the same rock material in where the measurements will be performed), and finally the core cell incorporates in addition one temperature transducer.

The data acquisition system module incorporates an electronic compass, multiplexers, a memory card, a communication port, some led, a stopwatch trigger, a main circuit that performs the control tasks of: computing, storing and transmitting, and an energy supplier.

A biaxial cell to assess the elastic variables of the rock on cylindrical HX samples was also designed to make possible calculations.

Final notes

The Furnas Type II Stress Measurement Cell was only possible with the help of al lot of persons that actuate efficiently at the different stages of the research. To tell this history in a compressed way will always deal in the involuntary error to omit one or more of those persons. But, from this report authors' opinion, was Doctor Armelin the principal person of the good results of this research.

Interesting documents about the Furnas Type II Stress Measurement Cell and its development may be found in the following references:

Armelin, J.L., 2010. In-situ stress measurement in rock masses and concrete structures (In Portuguese). PhD Thesis, Document G.TD-066/2010, Department of Civil and Environmental Engineering, University of Brasilia, Brasília, DF, p: 305.

Armelin, J.L., Fleury, S.V., Ruggeri, R.F., Piovezani, J.D., 2006. Laboratory validation of a triaxial cell for the in-situ stress assessment (In Portuguese). Proceedings of the 13th Brazilian Congress of Soil Mechanics and Geotechnical Engineering, Curitiba, Paper SMB001.

Armelin J.L., Fleury, S.V., Assis, A.P., 2004. Development of a triaxial cell for in-situ stress assessment (In Portuguese). Proceedings of the First Brazilian Congress of Tunnels and Underground Structures, SAT 2004, São Paulo (March): Article 19.

Armelin, J.L., Matos, M.M., Caproni Jr., N., 1994. Comparative analysis of instrumentation results for natural stress measurements in rock masses (In Portuguese). Proceedings of the First Brazilian Symposium on Rock Mechanics, Foz de Iguaçú, pp. 3-9.

Martins de Matos, M., Armelin, J.L., 1994. Natural stresses of the Serra da Mesa rock mass (In Portuguese). Proceedings of the First Brazilian Symposium on Rock Mechanics, Foz de Iguaçú, pp. 93-99.

http://www.isrm.net/fotos/editor2/nl15/furnastype2stresscell.pdf

ΔΙΑΚΡΙΣΕΙΣ ΕΛΛΗΝΩΝ ΓΕΩΤΕΧΝΙΚΩΝ ΕΠΙΣΤΗΜΟΝΩΝ

EUROCK 2013
ISRM European Regional Symposium
Rock Mechanics for Resources, Energy and Environment
23-26 September 2013, Wroclaw, Poland

Το μέλος της ΕΕΕΕΓΜ **Αλέξανδρος Σοφιανός**, Καθηγητής στη Σχολή Μηχανικών Μεταλλείων – Μεταλλουργών ΕΜΠ και Διευθυντής του Εργαστηρίου Τεχνολογίας Διάνοιξης Σηράγγων προσεκλήθη από τον Πρόεδρο της Polish Society for Rock Mechanics να συμμετάσχη στην Συμβουλευτική Επιτροπή του επομένου ISRM International Symposium EUROCK 2013:

Dear Professor Sofianos,

It is my pleasure to extend an invitation to you to join, as a representative of the ISRM National Group GREECE, the International Advisory Committee of the ISRM International Symposium EUROCK 2013 that will be held in the city of Wrocław on September 21-26, 2013. Prof. D. Łydżba, Chairman of the Organizing Committee, and I would very much appreciate your cooperation and support in our efforts to make EUROCK 2013 a success. We would be honored if you would accept our invitation and share your experience and expertise with us.

With the best wishes for a happy and prosperous New Year,

Marek Kwaśniewski

President, Polish Society for Rock Mechanics
Chair, Scientific Committee of the 2013 ISRM International Symposium

ΘΕΣΕΙΣ ΕΡΓΑΣΙΑΣ ΓΙΑ ΓΕΩΤΕΧΝΙΚΟΥΣ ΜΗΧΑΝΙΚΟΥΣ

Όπως όλοι γνωρίζουμε, η τρέχουσα οικονομική κατάσταση της χώρας έχει επηρεάσει σημαντικώτατα τον κατασκευαστικό κλάδο, με αποτέλεσμα πολλοί συνάδελφοι να έχουν απολυθή από τις εταιρείες όπου εργάζοντο ή να έχουν ελάχιστη απασχόληση. Δεδομένου ότι οι προοπτικές βελτίωσης της κατάστασης είναι ελάχιστες, τουλάχιστον στο άμεσο μέλλον, φαίνεται ότι η εκτός Ελλάδας απασχόληση είναι μία λύση. Για το λόγο αυτό ήρθαμε σε επαφή με αδελφές εθνικές επιστημονικές εταιρείες, προκειμένου να μεσολαβήσουμε αφ' ενός για την προώθηση βιογραφικών σημειωμάτων Ελλήνων γεωτεχνικών μηχανικών σε ενδιαφερόμενες εταιρείες στις χώρες τους, αφ' ετέρου δε για την ενημέρωση των μελών μας για διαθέσιμες θέσεις εργασίας.

Η ανταπόκριση ήταν άμεση!

Από τον καθηγητή Harry Poulos από την **Αυστραλία** λάβαμε το παρακάτω ηλ.μη.:

Coffey Geotechnics is interested in the possibility of hiring good quality geotechnical engineers to work in Australia, New Zealand or Canada. I would therefore be grateful if you could forward on to me any suitable CVs that you may have.

With best wishes,

Harry

PROFESSOR HARRY POULOS
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Από τον Arsenio Negro της εταιρείας BUREAU DE PROJETOS E CONSULTORIA LTDA της **Βραζιλίας** λάβαμε το παρακάτω ηλ.μη.:

Definitely we can try to help you on this. May I suggest you to prepare a teaser ad to be published in our home page (http://www.abms.com.br/) offering your support to find prospective geotechnical engineers in Greece for interested Brazilian companies. Try to draft one advertisement and mind that I can help you on this. Make it with high spirits and a dash of humor if possible!

Από την εταιρεία KLOHN CRIPPEN BERGER του **Καναδά** λάβαμε το παρακάτω ηλ.μη.:

Dear Dr. Tsatsanifos,

Your e-mail to your colleagues at the Canadian Geotechnical Society regarding the possibility of Greek Geotechnical Engineers to work in Canada was forwarded to me by Bryan Watts.

We are always looking for experienced engineers to join our organization (both locally and internationally).

As all of our current job vacancies are posted on-line, we invite your members to visit our career website (www.klohnjobs.com) to inquire about opportunities with our organization.

Please feel free to contact me should you have any questions.

Kind Regards,

Evelyn.

Evelyn Collier, CHRP
Senior Human Resources Advisor

Klohn Crippen Berger

Παρακαλούνται οι ενδιαφερόμενοι είτε να στείλουν στην ηλ.δι. president@hssmge.gr βιογραφικά τους σημειώματα για προώθηση ή να επικοινωνήσουν απ' ευθείας με τις ενδιαφερόμενες εταιρείες.

Δυστυχώς, όμως, υπήρξαν και αρνητικές απαντήσεις. Φαίνεται ότι το πρόβλημα απασχόλησης των μηχανικών, γενικώτερα, δεν είναι μόνο Ελληνικό! Από τον καθηγητή Robert Holtz από τις **Η.Π.Α.** λάβαμε το παρακάτω ηλ.μη.:

I am not sure the USNS can help Greek engineers, as much as we might like to do so. We have our own problems here with reduced spending on infrastructure and commerical developments. Many CEs here are also out of work...

ΠΡΟΣΕΧΕΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΚΔΗΛΩΣΕΙΣ

Ε Π Ε Ι Γ Ο Ν

18th International Conference on
Soil Mechanics and Geo-technical Engineering
"Challenges and Innovations in Geo-technics"
1 – 5 September 2013, Paris, France

Letter to the Member Societies of ISSMGE

10 November 2011

Subject: call for papers for the 18th International Conference on Soil Mechanics and Geotechnical Engineering (ICSMGE'13), Paris, France, 2-5 September 2013

Dear Presidents and Secretaries of ISSMGE Member Societies,

Dear Colleagues,

On behalf of the Organizing Committee for the 18th ICSMGE, I would like to inform you that the call for abstracts is now open.

According to the ISSMGE by-laws, the abstracts are selected by the Member Societies. The one page abstracts will be written in English or in French. The instructions for abstracts will be available on the Conference website and are attached to this message. Please note that the website (www.paris2013-icsmge.org) also provides a management tool for submission and acceptance of abstracts as well as of final papers. It can be used by authors for submission to their Member Society and by the Member Society to review abstracts and papers.

Selected abstracts have to be submitted by each Member Society to the Organizing Committee through the Conference website by 15 April 2012.

Although the Member Societies are requested to review the abstracts before submission, the abstracts will also be reviewed by the Scientific Committee of the Conference after submission by the Member Societies. Notification of the abstracts final acceptance will follow shortly.

Each Member Society is allocated a number of pages in the Proceedings following established ISSMGE rules. The number of pages will be indicated by the Secretary General of ISSMGE. All final papers should be four pages long. These indications will allow you to determine the number of abstracts for your Member Society.

After acceptance of the abstracts by the Organizing Committee, the Member Societies will be requested to collect the final four pages papers. After reviewing the papers, the Member Societies will transfer them to the Organizing Committee through the Conference website before 15 January 2013. The instructions for full papers will be available on the website. The final acceptance of the papers will be notified to the authors by 15 April 2013.

If you have questions, please contact us. We thank you very much in advance for your kind assistance and we look forward to receiving your abstracts by 15 April 2012.

With best regards

Philippe Mestat
President of the Organizing Committee

25 November 2011
Ref: C18

XVIII ICSMGE, Paris 2013 - Page Allocation

Page allocations for the XVIII ICSMGE to be held in Paris, September 1-5, 2013, have been made in accordance with our normal practice subject to the minor modification agreed at the Council Meeting in 2009. The total number of pages to be allocated is approximately 3000, and allocations have been made in multiples of 4 since the conference organisers are expecting each paper submitted to be 4 pages in length.

The allocation for your Member Society is 32 pages.

It is the responsibility of each Member Society to select its own papers for submission to the Conference Organising Committee and please note that Authors of papers ***must*** be members of ISSMGE. Please read the accompanying letter from the President of the Organising Committee, which has detailed information on deadlines and the paper submission and review process.

Finally, the President wishes you to note that he is prepared to consider allocating additional pages for papers related to engineering practice concerning projects or issues of major importance. I would appreciate prompt notification of your wish to apply for such additional page allocation.

Yours sincerely

Professor R.N. Taylor
Secretary General, ISSMGE
City University
Northampton Square
London EC1V 0HB. UK
Tel: +44 20 7040 8157
Fax: +44 20 7040 8832
E-mail: secretary.general@issmge.org

Προσκαλούνται τα μέλη της ΕΕΕΕΓΜ να ενημερώσουν (president@hssmge.gr) μέχρι την 20^{η} Ιανουαρίου 2012 για την πρόθεσή τους να υποβάλουν περίληψη άρθρου. Σε περίπτωση που οι προτιθέμενοι να υποβάλουν άρθρα υπερβαίνουν τους οκτώ (8), έχουμε την δυνατότητα να ζητήσουμε την διάθεση στην ΕΕΕΕΓΜ περισσοτέρων σελίδων από αυτές που της αναλογούν. Η σχετική αίτηση προς τον Πρόεδρο και τον Γενικό Γραμματέα της ISSMGE θα πρέπει να σταλή μέχρι την 23η Ιανουαρίου 2012.

Για τις παλαιότερες καταχωρήσεις περισσότερες πληροφορίες μπορούν να αναζητηθούν στα προηγούμενα τεύχη του «περιοδικού» και στις παρατιθέμενες ιστοσελίδες.

Spritzbeton - Tagung 2012, 2. und 13. 1. 2012, Alpbach, Tyrol, Austria, www.spritzbeton-tagung.com

Ετήσια Επιστημονική Συνεδρία ΕΓΕ, 6 Φεβρουαρίου 2012, Αθήνα, http://www.geosociety.gr

3ο ΠΑΝΕΛΛΗΝΙΟ ΣΥΝΕΔΡΙΟ ΟΔΟΠΟΙΙΑΣ
9 – 10 Φεβρουαρίου 2012, Αθήνα

Το Τεχνικό Επιμελητήριο Ελλάδας θα πραγματοποιήσει το «3ο Πανελλήνιο Συνέδριο Οδοποιίας» στην Αθήνα στις 9 και 10 Φεβρουαρίου 2012.

Σκοπός του Συνεδρίου είναι να παρουσιασθούν οι νεότερες εξελίξεις στον τομέα της οδοποιίας πάνω στην κατασκευή, ασφάλεια, συντήρηση και διαχείριση των οδικών έργων.

Με το Συνέδριο αυτό επιδιώκεται η ενημέρωση του τεχνικού κόσμου επί των παραπάνω θεμάτων, ιδιαίτερα αυτή την περίοδο που βρίσκεται σε εξέλιξη η υλοποίηση ενός εκτεταμένου προγράμματος Οδοποιίας στη χώρα μας.

Το Συνέδριο απευθύνεται στους Μηχανικούς, στα Ανώτατα Εκπαιδευτικά Ιδρύματα, στους Δημόσιους φορείς, στους Οργανισμούς, στις εταιρείες μελετών και γενικότερα σε όσους έχουν ιδιαίτερο ενδιαφέρον για τα θέματα της Οδοποιίας.

Θέματα του Συνεδρίου:

1. Γεωμετρικός Σχεδιασμός
2. Εθνικό Δίκτυο – Αυτοκινητόδρομοι
3. Τεχνικοοικονομική Σκοπιμότητα – Συστήματα Χρηματοδότησης – Παραχώρηση – Εκμετάλλευση
4. Διαχείριση – Δημοπράτηση – Ανάθεση Έργων
5. Οδική Ασφάλεια
6. Εξοπλισμός Οδών και Σηράγγων – Σύγχρονα Συστήματα Πληροφορικής
7. Συστήματα Διαχείρισης, Λειτουργίας και Συντήρησης Οδών
8. Έλεγχος και Διασφάλιση Ποιότητας Έργων
9. Τεχνικά Έργα Οδοποιίας – Νεότερες Εξελίξεις
10. Περιβαλλοντικές Επιπτώσεις

Εκτός από τις εισηγήσεις, στη διάρκεια κάθε ενότητας θα προσφέρεται χρόνος για συζήτηση και κατάθεση απόψεων ή προτάσεων σε όσους ασχολούνται ή έχουν άποψη για το θέμα της εκάστοτε ενότητας. Μέσα από έναν ανοιχτό και γόνιμο διάλογο, αναμένεται να προκύψουν τεκμηριωμένα επιστημονικά συμπεράσματα για τις προοπτικές όλων των σχετικών θεμάτων.

Φορείς στόχευσης

Το Συνέδριο προσφέρει την ευκαιρία σε ιδιωτικούς και δημόσιους φορείς, τοπική αυτοδιοίκηση, πανεπιστήμια, ερευνητικά κέντρα, ερευνητές, μελετητές, κατασκευαστές, άλλους επαγγελματίες να ανταλλάξουν απόψεις στην εξέταση προβλημάτων και λύσεων, νέων τεχνολογιών, σύγχρονων εξελίξεων σε θέματα Οδοποιίας.

Δικαίωμα Συμμετοχής: 50 €

Πληροφορίες: Νοτίνα Κοντογιάννη notina@central.tee.gr

4th International Conference on Grouting and Deep Mixing, February 15-18, 2012, New Orleans, Louisiana, USA, www.grout2012.org

28 - 29 February, 2012, Hong Kong, China
http://www.tunneldesignconstruction.com/Event.aspx?id=635336

Developing underground infrastructure for transportation, conveyance of water and wastewater, utility services and for commercial/mixed use purposes has become indispensable for Asia.

All urban centers of Asia have already installed extensive plans for underground infrastructure to cope with the shrinking and high value surface area. But tunnelling is one of the most complex and risky engineering activity.

The 3rd annual **Tunnel Design & Construction Asia** is a platform for tunnelling industry stakeholders to discuss and share best practices in constructing efficient and sustainable tunnel infrastructure.

Tunnel Design & Construction Asia 2012 will examine:

- Master plans and upcoming projects for road/rail, water/wastewater and utility tunnel infrastructure
- Funding, legal and contractual requirements
- Best practices in geotechnical investigations and instrumentation
- Urban tunnelling techniques
- Optimal selection of construction procedures in NATM and Mountain tunnelling
- TBM selection procedures for different soil/rock conditions
- Handling technical and construction contract risks
- Fire safety and ventilation design for long and immersed tunnels

The following trends will also be presented:

- Design and construction of shafts and portals
- Safety codes and procedures in tunnel construction
- Handling squeezing ground conditions during construction
- Effective use of geotechnical baseline reports
- Retrofit and upgrade of existing tunnels

For programme, registration, sponsorship and media enquiries, please contact enquiry@iqpc.com.sg

3rd International Seminar on Earthworks in Europe, 19 – 20 March, 2012, Berlin, Germany, www.fgsv.de/veranstaltungen_international.html?&tx_julleevents_pi1[showUid]=85&cHash=4153b585bc

Practices and Trends for Financing and Contracting Tunnels and Underground Works, 22-23 March 2012, Athens, www.tunnelcontracts2012.com

6th Colloquium "Rock Mechanics - Theory and Practice" with "Vienna-Leopold-Müller Lecture", 22-23 March 2012, Vienna, Austria, christine.cerny@tuwien.ac.at

GeoCongress 2012 State of the Art and Practice in Geotechnical Engineering, Oakland, California, USA, March 25-29, 2012, www.geocongress2012.org

UNDER CITY Colloquium on Using Underground Space in Urban Areas in South-East Europe, April 12 - 14, 2012, Dubrovnik, Croatia, www.undercity2012.com

Conference on the Mechanical Behavior of Salt, 16 – 19 April 2012, MINES ParisTech, Paris, France, www.saltmech7.com

TERRA 2012 XIth International Conference on the Study and Conservation of Earthen Architecture Heritage, 22 – 27 April 2012, Lima, Peru, http://congreso.pucp.edu.pe/terra2012/index.htm

Underground Infrastructure & Deep Foundations, 22 - 24 April, 2012, Jeddah, Saudi Arabia, Middle East, www.undergroundfoundationsaudi.com

3rd International Conference on Shaft Design and Construction 2012, 24 - 26 Apr 2012, London, UK, www.iom3.org/events/sdc2012

GEOAMERICAS 2012 II Pan-American Congress on Geosynthetics, Lima, Perú, 2 - 5 May 2012 www.igsperu.org

16th Nordik Geotechnical Meeting, 9-12 May, 2012, Copenhagen, Denmark www.ngm2012.dk

Second Southern Hemisphere International Rock Mechanics Symposium SHIRMS 2012, 14–17 May 2012, Sun City, South Africa, www.saimm.co.za

ITA-AITES WTC 2012 "Tunnelling and Underground Space for a global Society", Bangkok, Thailand, 18 to 23 May, 2012, www.wtc2012.com

Fifth International Symposium on Contaminated Sediments: Restoration of Aquatic Environment, May 23 - 25 2012, Montreal, QC, Canada, www.astm.org/SYMPOSIA/filtrexx40.cgi?+-P+EVENT_ID+1857+/usr6/htdocs/astm.org/SYMPOSIA/callforpapers.frm

EUROCK 2012 - ISRM European Regional Symposium - Rock Engineering and Technology, 27 – 30 May 2012, Stockholm, Sweden, www.eurock2012.com.

SECOND INTERNATIONAL CONFERENCE ON PERFORMANCE-BASED DESIGN IN EARTHQUAKE GEOTECHNICAL ENGINEERING, May 28-30, 2012, Taormina, Italy, www.associazionegeotecnica.it

INTERNATIONAL SYMPOSIUM & SHORT COURSES TC 211 IS-GI Brussels 2012 Recent Research, Advances & Execution Aspects of GROUND IMPROVEMENT WORKS, 30 May – 1 June 2012, Brussels, Belgium, www.bbri.be/go/IS-GI-2012

12th Baltic Sea Geotechnical Conference "Infrastructure in the Baltic Sea Region", Rostock, Germany, 31 May – 2 June, 2012, www.12bsgc.de

80th Annual Meeting - 24th ICOLD Congress, June, 2nd to 5th, 2012 - June, 6th to 8th, 2012, Kyoto, Japan, http://icold2012kyoto.org/

ISL 2012 NASL 11th International Symposium on Landslides, 3 ÷ 8 June 2012, Banff, Alta, Canada, corey.froese@ercb.ca, www.ISL-NASL2012.ca

International Symposium on Sustainable Geosynthetics and Green Technology for Climate Change (SGCC2011), which also serves as the Retirement Symposium of Prof. Dennes T. Bergado, 20 and 21 June 2012, Bangkok, Thailand, www.set.ait.ac.th/acsig/sgcc2011

46th U.S. Rock Mechanics Geomechanics Symposium, Chicago, USA, 24 – 27 June 2010, www.armasymposium.org

XII International Symposium on Environmental Geotechnology. Energy and Global Sustainable Development "Unveiling the Pathways to Global Sustainability", Los Angeles, USA, June 27 – 29, 2012, www.isegnet.org/2012/

ASTM Symposium on Dynamic Testing of Soil and Rock: Field and Laboratory, June 28 - 29 2012, San Diego, CA, USA, www.astm.org/D18symp0612.htm

Protection and Restoration of the Environment XI July 3-6, 2012, Thessaloniki, Greece, www.pre11.org

Shaking the Foundations of Geo-engineering Education, International Conference on Geotechnical Engineering Education, 4-6 July 2012, NUI Galway, Galway, Ireland, bryan.mccabe@nuigalway.ie

ANZ 2012 "Ground Engineering in a Changing World" 11th Australia-New Zealand Conference on Geomechanics, Melbourne, Australia, 15-18 July 2012, www.anz2012.com.au

A Symposium on EXPERIMENTAL STUDIES WITH GEOSYNTHETICS In Conjunction with 15th INTERNATIONAL CONFERENCE ON EXPERIMENTAL MECHANICS (ICEM15), Porto, Portugal, July 22-27, 2012, http://paginas.fe.up.pt/clme/icem15

Geotechnique Themed Issue 2012 "Offshore Geotechnics", www.geotechnique-ice.com

34th International Geological Congress 5 ÷ 15 August 2012, Brisbane, Australia, http://www.ga.gov.au/igc2012

2nd SASORE
SOUTH AMERICAN SYMPOSIUM ON
ROCK EXCAVATION
August 7-9, 2012. Costa Rica
Ramada Plaza Herradura Hotel

7 – 9 August 2012, San Jose, Costa Rica
www.civiles.org/acg/simposio

The Second South American Symposium on Rock Excavations, 2nd SASORE, will take place in Costa Rica. The event will be from 7th to August 9th, 2012. In this opportunity, this event is the only regional experience sponsored by the

ISRM. The Symposium, as well as other events of 2012, will be part of the Fiftieth Anniversary of ISRM. The National Group Costa Rica, part of the Geotechnical Costa Rican Association is in charge of organizing the 2nd SASORE.

The objective of the 2nd SASORE is to establish the exchange of knowledge and results of the new technologies application and to discuss recent experiences in rock excavation. The activity includes researchers, designers and builders from the South American Region, Central America and the Caribbean. Also, we count with the contribution of experts from the region and other countries.

The Symposium will have three main speakers from Europe. They will be in charge of several sessions. First, Professor Manuel Romana will develop a lecture about the issue of rock slope stability. Then, Professor Piergiorgio Grasso will be addressing the issue of underground excavations and finally, Professor Arno Zang will discuss the issue of in situ stress.

Three main speakers from South American will join the event during the 2nd SASORE. Professor Tarsicio Celestino will deliver a lecture about the rock mass characterization. Next, Professor Carlos Carranza Torres will discuss about the modeling and numerical analysis of rock excavation. Afterwards, Professor Milton Kanji will be in charge of the lecture about soft rocks and associated problems. Finally, the Professor Duncan Wyllie, from the North American Region, will deliver the lecture about rock foundation.

The participants of the 2nd SASORE will have the opportunity to experience the importance of the industry through the ExpoRock, where they will have access to the main information from expert companies.

According to the 1st newsletter that will circulate in January 2012, the last date for submission of abstracts is March 30th, 2012. The date for full paper submission is May 30th, 2012. For further details including fees for the participants, please visit the website www.civiles.org/acg/simposio.

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EYGEC 2012 Gothenburg 22nd European Young Geotechnical Engineers Conference, Gothenburg, Sweden, August 26th to 29th, 2012, www.sgf.net

ICSE-6, 6th International Conference on Scour and Erosion, 27-31 August 2012, Paris, France, www.icse-6.com

ISSMGE TC-101 Advances in Multiphysical Testing of Soils and Shales

ISSMGE Workshop
3-5 September 2012, Lausanne, Switzerland
http://amtss.epfl.ch

The workshop will focus on the signi_cant advances of knowledge regarding the experimental analysis of soils and shales that have been achieved during the last decade. Some fundamental issues have been solved, and important achievements have been made in certain areas, including the development of multiphase testing facilities for non-isothermal conditions and the characterization of the micro-structural arrangement for complex geomaterials.

This outstanding progress in the _eld has had relevant consequences in the theoretical developments of geomechanical theories, such as the constitutive modelling of multiphysical and multiscale processes, as well as important engineering applications.

The workshop is aimed at stimulating the debate on the advances in experimental geomechanics; contributions on unsaturated soil testing, non-isothermal experiments and chemo-osmotic experimental evidences are welcomed. The workshop proceedings will be published and an international journal special issue will include the most outstanding contributions. The foreseen duration is 2.5 days. A half-day course will also be organized on advanced multiphysical testing for geomaterials A low registration fee has been set for students to encourage young delegates to attend.

The workshop will be held between 3 and 5 September 2012 at the conference facilities of the EPFL in Lausanne (Switzerland). The Laboratory for Soil Mechanics (LMS) at the EPFL will serve as the local organizing committee.

Contacts:
Laboratory for Soil Mechanics
EPFL-ENAC-LMS
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Switzerland

Workshop secretary:
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Phone: +41 21 693 23 15
Fax: +41 21 693 41 53
E-mail: lms@epfl.ch
Web: http://lms.epfl.ch/en

Further information is available on the workshop website: http://amtss.epfl.ch

Baltic Piling Days 2012, Tallinn, Estonia, 3-5th September 2012, www.balticpiling.com

2nd International Conference on Transportation Geotechnics, 10 - 12 September 2012, Sapporo, Hokkaido, Japan, http://congress.coop.hokudai.ac.jp/tc3conference/index.html

7th International Conference in Offshore Site Investigation and Geotechnics: Integrated Geotechnologies, Present and

Future, 12-14 September 2012, London, United Kingdom, peter.allan@geomarine.co.uk; zenon@tamu.edu

CRETE 2012
3rd International Conference on
Hazardous and Industrial Waste Management
September 12 - 14, 2012, Chania, Greece
www.hwm-conferences.tuc.gr

The Conference will once again focus on innovative aspects of Industrial and Hazardous Waste Management (including Organics, Non-Halogenated and Halogenated Solvents, Hydrocarbons, Pesticides, Explosives, PCBs, PCDDs/Fs, Heavy Metals, Asbestos, Nuclear Wastes, Salts, etc), presenting new technologies, describing the state of the art and related case studies, discussing the main controversial subjects, sharing experience among different countries, valuating social and financial balances. The Conference will include oral presentations, poster sessions, special sessions and workshops.

CONFERENCE TOPICS

- Industrial and Hazardous Waste
- Regulation / Legislation
- Characterization
- Management Practices
- Production, Minimization and Recycling
- Treatment and Disposal
- Hazardous Waste Toxicology
- Risk Assessment
- Treatment of Hazardous Waste landfill and Mine Leachates
- Contaminant Release and Transport
- Toxic Substances in the Food Chain
- Management of Contaminated Sites
- Special Topics on Environmental Management and Remediation (sediment sites - characterization and risk assessment, munition and explosives production sites, etc.)
- Radioactive Waste (management, environment, health and safety, nuclear explosions, etc.)
- Energy From Waste (biomass, oil sludge, gasification processes, syngas, etc.)
- Case Studies
- Special Waste (medical, WEEE, agro-industrial, etc.)

Other topics of the "CRETE 2012" Conference are the following:

- Industrial and Hazardous Waste (Regulation / Legislation, Management Practices, Production, Minimization and Recycling, Treatment and Disposal)
- Hazardous Waste Toxicology - Risk Assessment
- Treatment of Hazardous Waste Landfill and Mine Leachates
- Contaminant Release and Transport
- Toxic Substances in the Food Chain
- Management of Contaminated Sites
- Special Topics on Environmental Management and Remediation (Sediment Sites – Characterization and Risk Assessment, Munition and Explosives Production Sites, etc.)
- Radioactive Waste (Management, Environment, Health and Safety, Nuclear Explosions, etc.)
- Energy from Waste (Biomass, Oil Sludge, Gasification Processes, Syngas, etc.)
- Case Studies
- Special Waste (Medical, WEEE, Agro-Industrial, etc.)

The following Special Workshops have been planned:

Characterization, Restoration and Management of Refineries

Refineries represent common heavy industrial sites that develop important soil and groundwater contamination problems, mainly due to uncontrolled leaks. The management of such contaminated sites, especially when the refineries are under operation, is a complicated issue that demands specialized knowledge, experience and technolgy. During this "CRETE 2012" Special Workshop, case studies, as well as optimum management and restoration approaches, are to be presented.

Mines Restoration and Mines Leachates Treatment

Mining activities can be considered as heavy industrial activities that may bear severe risk and hazardousness, depending on their paricular characteristics. Leachates treatment, hazardous materials management, safety measures, as well as abandonded mines restoration are some of the basic topics that are expected to be explored and presented during this "CRETE 2012" Special Workshop.

Asbestos Materials/Waste Management

Asbestos has been allocated at many industrial facilities, due to its special properties that allowed the production of several asbestos containg products. Since the exposure of its hazardousness and the banning of its use, research has focused on managing abandoned asbestos mines and facilities. Moreover, a lot of attention has been given to the withdrawal, treatment and disposal of asbestos containing materials. During this "CRETE 2012" Special Workshop, the prevailing conditions worldwide, as well as the latest scientific and technical advances, on asbestos management are attempted to be presented.

Authors are invited to **submit relevant abstracts**, to be presented during these Special Workshops. An extended abstract (at least one, but no more than two full pages), accompanied by the relative Abstract form, should reach the Conference Organization no later than **31st January 2012**.

More information is available at our official website: http://www.hwm-conferences.tuc.gr/

EUROGEO5 - 5th European Geosynthetics Conference, 16 - 19 September 2012, Valencia, Spain, www.eurogeo5.org

IS-Kanazawa 2012 The 9th International Conference on Testing and Design Methods for Deep Foundations 18-20 September 2012, Kanazawa, Japan, http://is-kanazawa2012.jp

ISC' 4 4th International Conference on Geotechnical and Geophysical Site Characterization, September 18-21, 2012, Porto de Galinhas, Pernambuco – Brazil, www.isc-4.com

1st Eastern European Tunneling Conference, September 18-21, 2012, Budapest, Hungary, www.eetc2012budapest.com

The 4th Central Asian Geotechnical Symposium: Geo-Engineering for Construction and Conservation of Cultural Heritage and Historical Sites Challenges and Solutions September 2012 Samarkand, Uzbekistan

The Uzbek Geotechnical Society takes great pleasure in inviting all members and Member Societies of ISSMGE Asian region and interested geotechnical engineers and researchers from all over the world to participate in the Conference on Geo-Engineering for Construction and Conservation of Cultural Heritage and Historical Sites of Cultural Heritage and Historical sitesto be held in September 21-23, 2012 in Samarkand, Uzbekistan.

The previous Central Asian Geotechnical Symposiums was in Astana, Kazakhstan, 2000; Samarkand, Uzbekistan, 2003; Dushanbe, Tajikistan, 2006.

The 4thCentral Asian Geotechnical Symposium intends to discuss and exchange ideas on general characteristics of soils in the region, geotechnical problems as well as conservation of heritage and historical sites.

Samarkand ("Stone Fort" or "Rock Town") is the second-largest city in Uzbekistan and the capital of Samarqand Province. The city is most noted for its central position on the Silk Road between China and the West, and for being an Islamic center for scholarly study. In the 14th century it became the capital of the empire of Timur and is the site of his mausoleum. The Bibi-Khanym Mosque remains one of the city's most notable landmarks. The Registan was the ancient center of the city.

In 2001, UNESCO added the city to its World Heritage List as *Samarkand – Crossroads of Cultures*.

Theme and topics which are related of ATC19, ATC3, ATC10, TC301, and TC305 include:

1. Regional Characterization of Soils and Foundation, and Geo-Construction
2. Adobe, Tomb and Earthen Structures, Historical Sites, and Conservation of Cultural Heritage
3. Regional and Traditional Characteristics of Foundation and Structures
4. Mosque, Minaret, Towers, Citadel, Castles, Stone Masonry, and Heritage Structures
5. Ancient Caves, Underground Construction, Tunneling, Transportation, and Infrastructures
6. Ancient and Historical Dam, Embankment, and Ancient Highways
7. Soil Dynamics and Geotechnical Earthquake Engineering
8. Ancient, Traditional, and Present Soil Improvements
9. Damages from Salting and Frost including Geoenvironmental Engineering
10. Traditional and Innovative Technologies for Geotechnical Applications

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Fax: +998-66 237-00 16 Tel: +998-66 220-2825

SAHC 2011, 8th International Conference on Structural Analysis of Historical Constructions, October 15 – 17, 2012, Wroclaw, Poland, www.sahc2012.org

7th Asian Rock Mechanics Symposium, 15-19 October 2012, Seoul, Korea, www.arms7.com

HYDRO 2012

Innovative Approaches to Global Challenges
29 to 31 October 2012, Bilbao, Spain
www.hydropower-dams.com

HYDRO 2012 will again bring together the world hydropower community from about 80 countries to discuss potential and plans for future large and small-scale hydro development. Panellists will discuss investment in hydropower: new financing strategies; economic aspects; risk management; multipurpose benefits; and the public perception of hydropower. Technical presentations will reflect innovations in technology, design trends, safety, efficiency and economy in operation, and new engineering challenges. International collaboration and integrated regional development will be other key issues, along with environmental and social aspects, and climate change.

As policy-makers begin to take a more positive view of hydropower, and more energy and water 'roadmaps' are leading towards new hydro projects, HYDRO 2012 will make a contribution to the major task of turning policies into practice.

The topics below are indicators of the scope of the conference, but other related subjects may also be proposed:

- Potential and development needs
- Project financing
- Commercial and economic aspects
- Environmental and social aspects
- Multipurpose schemes
- Hydraulic and electrical machinery
- Hydro in synergy with other renewables
- The role and benefits of pumped storage

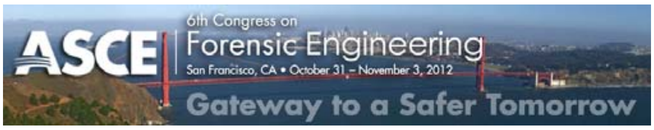
- System management
- Small hydro
- Developments in marine energy
- Upgrading of equipment and civil works
- The image of hydro
- Civil engineering
- Project management
- Sedimentation management.

Information about HYDRO 2012 will be updated regularly in *The International Journal on Hydropower & Dams* and on our website (www.hydropower-dams.com).

An important element of HYDRO 2012 will be the major international Technical Exhibition which will extend throughout the Congress Centre, alongside the conference rooms. About 240 companies active in the hydro and dams profession will demonstrate their expertise and scope of supplies or services. 80% of the exhibition space has been reserved already, so if you have not yet booked, reserve your space now!

For enquiries, contact:
HYDRO 2012 Conference Management Team
PO Box 285, Wallington, SM6 6AN, United Kingdom
Tel: +44 20 8773 7244; Fax: + 44 20 8773 7255; Email: Hydro2012@hydropower-dams.com

International Conference on Ground Improvement and Ground Control: Transport Infrastructure Development and Natural Hazards Mitigation, 30 Oct - 2 Nov 2012, Wollongong, Australia www.icgiwollongong.com



6th Congress on Forensic Engineering
October 31 – November 3, 2012, San Francisco, USA
http://content.asce.org/conferences/forensics2012/index.html

The Technical Council on Forensic Engineering, the Steering Committee, and Cooperating Organizations, are pleased to present the 6th Congress on Forensic Engineering to be held at the Hyatt Regency San Francisco, 5 Embarcadero Center, San Francisco, California on October 31, 2012 - November 3, 2012.

Forensic engineering is the application of engineering principles to the investigation of failures or other performance problems. Forensic engineering may also involve testimony on the findings of these investigations before a court of law or other judicial forum, when required. Failures are not all catastrophic, such as when a building or bridge collapses, but include facilities or parts of facilities that do not perform as intended by the owner, design professional or constructor.

Objectives:

The objectives of the Congress are to provide a forum for the exchange of:

1) practices and procedures to reduce the number of failures,
2) information on failures, their cause, and repair,
3) the state of practice and the process for conducting a forensic investigation, and
4) the state of practice for ethical conduct.

Who Should Attend:

Those who should attend include forensic engineers, civil engineers, architectural engineers, building owners, property managers, adjusters, attorneys, educators, students and invited guests. Registrants will benefit from an exchange of practices and procedures, educational opportunities, dissemination of information, opportunities for publishing, networking and will earn professional development credit.

Program:

The 6th Congress on Forensic Engineering will open with pre-Congress workshops held on Wednesday, October 31, 2012, to provide educational opportunities and refresher workshops to examine issues in depth in forensic engineering. This will be followed by a three-day technical program on Thursday through Saturday, November 1-3, 2012, with four concurrent technical tracks, committee meetings, panel discussions, demonstrations, exhibits, icebreaker reception, keynote address, awards luncheon, spouse and technical tours and networking opportunities. Networking and social events are planned to encourage networking opportunities among colleagues and other experts, adjusters, attorneys, educators, students and friends.

Topics:

- Building Codes
- Building Envelope / Buildings and Structures
- Purpose Structures
- Bridges
- Disaster Risk Management
- Critical Facilities Due to Tsunami
- Education
- Engineering Education
- Environmental
- Failures During Construction
- Forensic Investigations
- Geotechnical
- Legal
- Lifeline Earthquake Engineering
- Infrastructure
- Pipelines
- Professional Practice
- Repairs and Remediation
- San Francisco Projects
- Shock, Blast, and Impact (Extreme Loading)
- Special Topics
- Wind Engineering

ACUUS 2012 13th World Conference of the Associated Research Centers for the Urban Underground Space Underground Space Development – Opportunities and Challenges, 7 – 9 November 2012, Singapore, www.acuus2012.com

32. Baugrundtagung with exhibition "Geotechnik", Mainz, Germany, 26 – 29 November 2012

GEOSYNTHETICS ASIA 2012 (GA2012) 5th Asian Regional Conference on Geosynthetics, Bangkok, Thailand, 10 - 14 December 2012, www.set.ait.ac.th/acsig/igs-thailand

First International Congress FedIGS, 12 – 15 November 2012, Hong Kong – China, www.fedigs.org/HongKong2012

GA2012 - Geosynthetics Asia 2012 5th Asian Regional Conference on Geosynthetics, 10 - 14 December 2012, Bangkok, Thailand, www.set.ait.ac.th/acsig/GA2012

Fourth International Seminar on
FORENSIC GEOTECHNICAL ENGINEERING
January, 10-12, 2013, Bengaluru, India

Forensic geotechnical engineering involves scientific, legalistic investigations and deductions to detect the causes as well as the process of distress in a structure, which are attributed to geotechnical origin. Such a critical analysis will provide answers to "what went wrong, when, where, why, how, and by whom". Cases of remedied installations, particularly those which, fall under public / or government category, where the analysis and evaluation of adopted remedial measures with regard to their effectiveness and economy may be subjected to judicial scrutiny also, fall under this purview. It also gives strong inputs to improve future designs. The normally adopted standard procedures of testing, analysis, design and construction are not adequate for the forensic analysis in majority of cases. The forensic investigations involve fresh field and laboratory tests apart from collection of all available data. The test parameters and design assumptions will have to be representative of the actual conditions encountered at site. While the designs are mostly stress based, the forensic analysis has to be deformation based. The forensic geotechnical engineer (who is different than the expert witness) has to be not only thorough in his field of specialization, but also be familiar with legal procedures. This seminar highlights the principles of planning and executing a forensic investigation citing case histories.

The International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) constituted a Technical Committee (TC) on Forensic Geotechnical Engineering (FGE) in 2005. During the first four years it was designated as TC40 and is now designated as TC302. This committee has already conducted three international seminars on FGE and is now organizing the fourth seminar during January 10-12, 2013 at Bengaluru (formerly known as Bangalore), India. The theme of the seminar is "Forensic Geotechnical Engineering" and is divided into ten topics as given below. During this three days seminar, these ten topics will be discussed in ten sessions:

1. Collection of Data
2. Characterization of Distress
3. Development of Failure Hypothesis
4. Diagnostic Tests
5. Back Analysis
6. Observation Method of Performance Evaluation
7. Reliability Aspects
8. Legal Issues
9. Case Histories
10. Technical Shortcomings

All Correspondence relating to the seminar shall be sent to Prof. G L Sivakumar Babu, 4th ISFGE, Organizing Secretary, Department of Civil Engineering, Indian Institute of Science Bengaluru - 560 012. Email: isfge2013@gmail.com Phone: +91-80 2293 3124 / 2360, Fax: +91-80 2360 0404.

Geotechnical Special Publication, ASCE "Foundation Engineering in the Face of Uncertainty". Abstracts to Mohamad H. Hussein at: MHussein@pile.com.

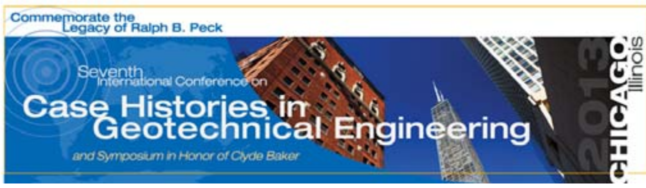
Geotechnical Special Publication, ASCE "SOUND GEOTECHNICAL RESEARCH TO PRACTICE", http://web.engr.oregonstate.edu/~armin/index_files/Holtz GSP

Themed Issue on Geotechnical Challenges for Renewable Energy Developments, Geotechnical Engineering 2013, ben.ramster@icepublishing.com

Pam-Am UNSAT 2013 First Pan-American Conference on Unsaturated Soils, 20-22 February 2013, Cartagena de Indias, Colombia, panamunsat2013.uniandes.edu.co

TU-SEOUL 2013 International Symposium on Tunnelling and Underground Space Construction for Sustainable Development, March 18-20, 2013, Seoul, Korea www.tu-seoul2013.org

Fifth International Conference on Forensic Engineering Informing the Future with Lessons from the Past, 15-17 April 2013, London, United Kingdom, http://ice-forensicengineering.com



Conference to Commemorate the Legacy of Ralph B. Peck, 7th International Conference on Case Histories in Geotechnical Engineering & Soil Dynamics and Symposium in Honor of Clyde Baker, Chicago, USA, 29 April – 4 May, 2013, http://7icchge.mst.edu

ITA-AITES WTC 2013 "Underground – the way to the future", Geneva, Switzerland, 10 to 17 May 2013, www.wtc2013.ch/congress

Effective and Sustainable Hydraulic Fracturing - an ISRM Specialized Conference
20-22 May 2013, Brisbane, Queensland, Australia
http://www.csiro.au/events/HF2013

HF2013 is co-organised and sponsored by CSIRO, the University of Utah's Energy and Geoscience Institute, the International Society for Rock Mechanics (ISRM) and the Australian Geomechanics Society (AGS).

During the conference, participants will explore and recognise the synergies and versatility of hydraulic fracturing in multiple industries, and promote its sustainable use as a key technology into the future.

The Conference will focus on three technical themes:

1. Advancing Effectiveness presenting the latest advances in simulation, theory, field procedures, laboratory experimenttation and case studies.

2. Exploring Versatility presenting methods and lessons from a diversity of application domains.

3. Promoting Sustainability driving toward differentiation between real and perceived risks, deployment of viable controls and beneficials public engagement.

Conference topics will include

- Preconditioning for mining
- Stress management
- CO_2 sequestration
- Naturally fractured reservoirs
- Coal seam gas
- Shale gas and shale oil
- High temperature, high stress reservoirs
- Water flooding
- Waste injection and remediation
- Engineered Geothermal Systems
- Environmental remediation
- Induced seismicity
- Magmatic dykes and sills
- Injection grouting
- Water well stimulation
- Monitoring and predicting growth
- Engaging communities.

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Second International Symposium on Geotechnical Engineering for the Preservation of Monuments and Historic Sites
30 -31 May 2013, Napoli, Italy
www.tc301-napoli.org

The conservation of monuments and historic sites is one of the most challenging problems facing modern civilization. It involves a number of factors belonging to different fields (cultural, humanistic, social, technical, economical, administrative), intertwining in inextricable patterns. In particular, the requirements of safety and use appear (and often actually are) in conflict with the respect of the integrity of the monuments. In almost all countries of the world the conservation is looked after by an official trained in Art History or Archaeology. He has generally the control of any action to be undertaken, and imposes constraints and limitations that sometimes appear unreasonable to the engineer. The engineer, in turn, tends to achieve safety by means of solutions which appear unacceptable to the official in charge of conservation, sometimes mechanically applying procedures and regulations conceived for new structures. It is evident that some equilibrium has to be found between the safe fruition of a monument and the respect of its integrity. The former task belongs to the know-how of any well trained and experienced engineer, while the latter one is more difficult, being the same concept of integrity rather elusive.

The difficulty of the problem is increased by the lack of a general theory, universally accepted and guiding the behaviour of the actors involved as the Mechanics does with the structural engineer. The possibility of finding in practice an acceptable equilibrium is linked to the development of a shared culture. The International Society of Soil Mechanics and Geotechnical Engineering contributed to this development by an ad hoc Committee (TC 19 – Conservation of Monuments and Historic Sites), that has been promoted over 25 years ago by French and Italian engineers (Jean Kerisel, Arrigo Croce). A number of international and regional symposia have been organised, always with large audience and lively discussions. A Lecture dedicated to Jean Kerisel will be given for the first time at the next International Conference on Soil Mechanics and Geotechnical Engineering to be held in 2013 in Paris. In this framework, the Technical Committee (now TC301) is organising the 2nd International Symposium on Geotechnical Engineering for the Preservation of Monuments and Historic Sites, which will be held in Napoli on May 2013. Its aim is that of comparing experiences, presenting important achievements and new ideas, establishing fruitful links.

The contributions to the Conference should focus on the following main themes:

1. Geotechnical aspects of historic sites, monuments and cities;
2. Past design criteria and traditional construction methods;
3. Techniques to preserve ancient sites and constructions;
4. Rehabilitation of heritage;
5. Role of geotechnical engineering in preservation of cultural and historical integrity.

Scientific secretariat

For general queries please contact:
info@tc301-napoli.org

For queries about paper submission please contact:
secretariat@tc301-napoli.org

or

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Emilio Bilotta (phone: +39 081 76 83469; email: emilio.bilotta@unina.it)

☙ ❧

STREMAH 2013 13th International Conference on Studies, Repairs and Maintenance of Heritage Architecture, 25 – 27 June 2013, New Forest, UK, carlos@wessex.ac.uk

☙ ❧

The 6th International Symposium on Rock Stress
20-22 August 2013, Sendai, Japan
http://www2.kankyo.tohoku.ac.jp/rs2013

The aims of the Symposium - RS2013 - are to discuss various problems and topics on rock stress and rock stress measurements for the benefit of exchanging knowledge to all the countries of the world. Contributions are welcome within civil, mining, geothermal, petroleum engineering and earth science.

Rock mechanics engineers, researchers and practitioners, as well as geologist, geophysicist and earth scientist are welcome.

Scope & Themes

The focus of the Symposium is to find a new trend in the field of rock stress and its measurement at a wide variety of depths for earth science and engineering. Topics of interest include, but are not limited to:

- Pactical approaches to in situ stress measuremnts
- Frontiers of in situ stress measurements
- Integration of stress data and methods
- Rock stress for deep mining
- Rock stress for energy source developments
- Tectonic stress and earthquakes.

Contact: info-rs2013@mail.kankyo.tohoku.ac.jp

☙ ❧

18th International Conference on Soil Mechanics and Geotechnical Engineering "Challenges and Innovations in Geotechnics", 1 – 5 September 2013, Paris, France www.paris2013-icsmge.org

Géotechnique Symposium in Print on Bio- and Chemo-Mechanical Processes in Geotechnical Engineering, www.elabs10.com/content/2010001471/SIP%202013.pdf

☙ ❧

EUROCK 2013
ISRM European Regional Symposium
Rock Mechanics for Resources, Energy and Environment
23-26 September 2013, Wroclaw, Poland

Contact Person: Prof. Dariusz Lydzba
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☙ ❧

ANDORRA 2014 14th International Winter Road Congress 2014, 4-7 February 2014, Andorra la Vella (Andorra), www.aipcrandorra2014.org

☙ ❧

EUROCK 2014
ISRM European Regional Symposium
Rock Engineering and Rock Mechanics: Structures in and on Rock Masses
26-28 May 2014, Vigo, Spain

Contact Person: Prof. Leandro Alejano
ETSI MINAS - University of Vigo
Dept. of Natural Resources & Environmental Engineering
Campus
Lagoas Marcosende
36310 Vigo (Pontevedra), SPAIN
Telephone: (+34) 986 81 23 74
E-mail: alejano@uvigo.es

☙ ❧

8th European Conference "Numerical Methods in Geotechnical Engineering", Delft, The Netherlands, 18-20 juni 2014, www.numge2014.org

10th International Conference on Geosynthetics – 10ICG, Berlin, Germany, 21 – 25 September 2014 www.10icg-berlin.com

13th ISRM International Congress on Rock Mechanics
Innovations in Applied and Theoretical Rock Mechanics
29 April – 6 May 2015, Montreal, Canada

The Congress of the ISRM "Innovations in Applied and Theoretical Rock Mechanics" will take place on 29 April to 6 May 2015 and will be chaired by Prof. Ferri Hassani.

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ΝΕΑ ΑΠΟ ΤΙΣ ΔΙΕΘΝΕΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΝΩΣΕΙΣ

International Society for Rock Mechanics

To: all ISRM National Groups
Date: 29 December 2011
Re: ISRM Young Members' Presidential Group

Dear Presidents of the ISRM National Groups

As you know, the new Board of the ISRM started its term of office last October and is now beginning to implement several measures that aim at improving the benefits the ISRM gives to its members.

One of these measures is the setting up of the ISRM Young Members' Presidential Group (YMPG). This group was originally established in 2010 and has now to be enlarged in order to make it more active and to address young members' interests. The YMPG shall be composed of one member appointed by each National Group, plus the ISRM President and Secretary General.

The main subjects to be addressed by the YMPG in the term of 2011-2015 are:

1. Issues they, and younger members in general, have concerning the Society, such as:
   - Creation of a student membership category and special fee,
   - Mentorship and assistance programs for young rock mechanics professionals (young members could for instance be automatically assigned a volunteer 'mentor'),
   - Organization of Regional Young Geotechnical Engineering Symposia,
   - Organization of Regional Rock Engineering Summer schools;
2. Initiatives for making the Society more responsive to the needs of young members;
3. Initiatives for addressing the objectives and purposes of the Society;
4. Injecting new ideas into the on-going modernisation plan;
5. Suggestions for promoting the Society to young rock mechanics professionals who are not members;
6. Initiatives for increasing the overall membership of the Society,
7. Developing a micro website linked to the ISRM website.

Each National Group is invited to appoint one young member, under 35 years of age, to join the YMPG. Please send to the Secretary General (secretariat.isrm@lnec.pt), latest by 31 January 2012, the details of your representative, including: name, date of birth, professional affiliation and email address.

Yours sincerely

President

Προσκαλούνται τα μέλη της ΕΕΕΕΓΜ να εκδηλώσουν το ενδιαφέρον του αποστέλλοντας σχετικό ηλ.μη. στην ηλ.δι. president@hssmge.gr

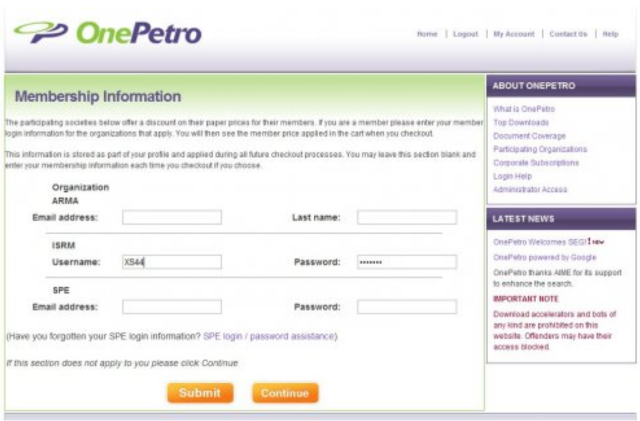
Access to the ISRM Digital Library with OnePetro

One of the advantages of the new Membership Management System is the improved access to the ISRM Digital Lybrary, which is hosted by OnePetro.

From January 3rd, the new ISRM usernames and passwords will be used to validade ISRM membership on the site OnePetro.org.

If you never tried this ISRM service, create an account with Onepetro using your email and a password of your choice. Then, once in the site, go to My Account and then click to update your membership information and use your new ISRM username and password to certify your ISRM member status. You will then be able to download up to 100 papers per year, from the many ISRM meetings already uploaded.

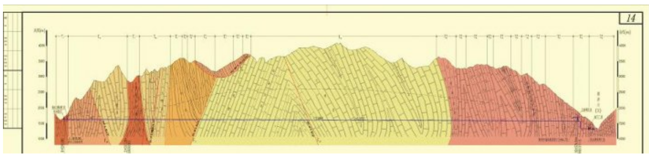
If you already use OnePetro, just make sure that you replace your ISRM old credentials by the new ones. Please remember, the new credentials will become active after January 3rd.



Jinping II - The world's largest tunnel group is broken through

On December 8th, 2011, the last one of the world's largest tunnel group is broken through at the job-site of Jinping II Hydropower Project on Yalong River, Sichuan, China. The project is a symbol in the great scheme of Western China Development and the power transmission from west to east. The tunnel group contains four power tunnels, one drainage tunnel and two traffic tunnels. The overall length is about 120km. The other three power tunnels were drilled through respectively on June 6th, August 16th, and October 20th.

The four power tunnels running through the mountain Jinping are in parallel. The average depth of the overburden is between 1,500m and 2,000m (with the maximal 2,525m), the average length of the tunnels from the intake to the surge chamber is 16.67km, and the excavated diameter changes from 12.4m (TBM boring section) to 13.0m (D&B section). These make it the world's largest and deepest tunnel group.



During four years construction since August 16th, 2007, a series of difficulties including long blind heading, sudden water inflow under high pressure, rock burst under high ground stress, and long tunnel ventilation were overcome. The success in fighting with all these world-class challenges is a huge progress in geotechnical engineering.

Wang Jimin and Jiao Kai
Ertan Hydropower Development Company, Ltd., Chengdu, Sichuan, China,610051.

SIMSG ISSMGE

TC 211

ISSMGE TC 211 GROUND IMPROVEMENT NEWSLETTER n°3, December 2011

Dear Reader,

As promised in our second Newsletter we would like to inform you about the on-going progress with the organization of IS-GI Brussels 2012 and the Short Courses on Wednesday 30 May 2012. This Newsletter contains the full programme of the 3 Short Courses.

1 Organizing Committee

After the first meeting in Lille (France) on 11 May 2011, other meetings of the Organizing Committee took place in Brussels on 17 June, 29 September and 7 December. So the Organizing Committee is working very hard to offer you all a very well organized symposium in Brussels next year. Below a picture taken during the last meeting on the 7th of December.

2 Sponsoring

The Organizing Committee acknowledges the support of 17 companies, subdivided as follows:

6 Platinum Sponsors

9 Gold Sponsors

2 Silver Sponsor.

It has been decided to limit the number of Platinum Sponsors to 6. So actually additional sponsoring is only possible for Gold and Silver. Sponsoring conditions can be found on the symposium website www.bbri.be/go/IS-GI-2012.

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3 Short courses

The programs for the 3 short courses on Wednesday 30th of May have been finalized and are given below. Please note that the program may be subject to minor changes. Participants will be informed accordingly.

The proceedings of the short courses will contain the powerpoint presentations.

Participants to the short courses will only receive the proceedings of the Short Course they are attending.

4 Symposium

All available information can be found on the symposium website www.bbri.be/go/IS-GI-2012

On-line registration is possible since begin November 2011.

The deadline for early-bird registration is 15 January 2012 !

SC 1: MARINE GROUND IMPROVEMENT

Coordinators:
M. Van Den Broeck, DEME, Belgium
S. Bretelle, GHD, Australia
Ph. Liausu, MÉNARD, France

PROGRAMME

09h30 – 10h00 : General Overview of Ground Improvement methods, *P. Mengé, DEME, Belgium*
10h00 – 10h30 : Overview of major geotechnical concerns, *M. Van Den Broeck, DEME, Belgium*
10h30 – 11h00 : Site investigations for Marine Ground Improvement, *S. Bretelle, GHD, Australia*

11h30 – 12h00 : Dynamic compaction and replacement, *Ph. Liausu, Ménard, France*
12h00 – 12h30 : Vibroflotation, *S. Lambert, Keller France*
12h30 – 13h00 : Rapid Impact Compaction, *J. Dykstra, COFRA, The Netherlands*

14h00 – 14h30 : Consolidation PVD, *I. Chu, Iowa State University , USA-Singapore*
14h30 – 15h00 : Vacuumconsolidation, *B. Indraratna, University of Wollongong, Australia*
15h00 – 15h30 : Soft Soil Improvement, *M. Van Den Broeck, DEME, Belgium*

16h00 – 16h30 : Reuse of dredged material, Project AMORAS
16h30 – 17h30 : Case studies, *P. Mengé and M. Van Den Broeck, DEME, Belgium*
17h40 – 17h55 : Discussion

18h00 – 20h00 : Reception

During the presentations special attention will be given to the specific issues for underwater ground improvement design and to monitoring and quality control (constraints and practical solutions).

SC 2 : DEEP MIXING

Coordinators:
J. Maertens, Jan Maertens BVBA & KU Leuven, Belgium
N. Denies and N. Huybrechts, Belgian Building Research Institute, Belgium

PROGRAMME

09h30 – 10h15 : General Overview, *A. Puppala , University of Texas, USA*
10h15 – 11h00 : Deep Mixing in contamined soils, *A. Al-Tabbaa, Cambridge University*

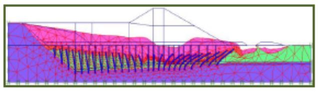
11h30 – 12h30 : Deep Mixing Equipment:
Bauer, *Germany*
Solétanche Bachy, *France*
Liebherr, *D. Quasthoff, Germany*
12h30 – 13h00 : Discussion

14h00 – 14h45 : Deep Mixing Research Belgian Building research Institute, *N. Denies*
14h45 – 15h05 : Deep Mixing Research KU Leuven, *A. Vervoort*
15h05 – 15h25 : Deep Mixing research Ghent University, *D Verastegui*
15h25 – 15h40 : Discussion

16h10 – 16h40 : Deep Mixing Design, *G. Filz, Virginia Tech, USA*
16h40 – 17h10 : Deep Mixing Monitoring, *Noel Huybrechts, BBRI, Belgium*
17h10 – 17h40 : Case study, *E. Leemans, ABEF & Soiltech, Belgium*
17h40 – 17h55 : Discussion

18h00 – 20h00 : Reception

SC3: RIGID INCLUSIONS AND SOIL REINFORCEMENT



Coordinators:
T. Durgunoglu, Zeats Zenin Teknolojisi A.A., Turkey,
B. Simon, TERRASOL, France
J. Sankey, The Reinforced Earth Company, USA

PROGRAMME

RIGID INCLUSIONS

09h30 – 10h00 : Lessons from centrifuge model tests on piled embankments and foundation over soil reinforced by inclusions, *L. Thorel, IFSTTar , France*
10h00 – 10h45 : Piled embankments reinforced with Geosynthetics : model experiments and analytical model developments, *S. Van Eekelen, Deltares,The Netherlands*

11h00 – 11h30 : US practice in Load Transfer Platform design : Lessons learned, *M. Walker, GEI consultants, USA*
11h30 – 12h05 : A review of the ASIRI research project findings about behaviour and design of foundations on soil reinforced by rigid inclusions, *B. Simon, Terrasol, France*
12h05 – 12h20 : Design of spread footing on soil reinforced by rigid inclusions, *B. Simon Terrasol, France*
12h20 – 12h55 : Design of slabs on grade supported with reinforced soil by rigid inclusions and Design assessment of limiting pressure at head of inclusions, *J. Racinais, Ménard, France*

REINFORCED SOIL

14h00 – 14h45 : Introduction to Mechanically Stabilized Earth (MSE) Wall Technology, *J. Sankey, the Reinforced Earth Company, USA*
14h45 – 15h30 : Review of Inextensible and Extensible MSE Reinforcement Applications
15h30 – 15h45 : Discussion

16h15 – 17h00 : Application of Local Country Codes for MSE Wall Design
17h00 – 17h45 : Combined MSE Wall Applications with Ground Improvement Technologies, *T. Durgunoglu, Zetas Zemin Teknolojisi A.S., Turkey*
17h45 – 18h00 : Discussion

18h00 – 20h00 : Reception

5 Other TC 211 related events

Through members of the TC 211 we received information on the following events:

- International Conference on Ground Improvement and Ground Control – Transport Infrastructure Development and Natural Hazard Mitigation, ICGI Wollongong 2012, 30 October – 2 November 2012, www.icgiwollongong.com. This Conference, on the Antipodes of Brusseles conference is co-sponsored by the TC 211, and belongs to the core activity of the TC 211. We will soon be requesting ideas and suggestions from all participants in the preparation of the S.O.A. report attributed to our TC211.

- Technical Session "Behavior and characterization of cemented and stabilized soils" at GeoCongress 2012 in March 2012, Ramiro.VerasteguiFlores@Urgent.be.

6 Next Newsletter

We will try to include in the next newsletter a list with recent references to publications and internet sites concerning the themes given in the terms of reference.

Please transmit all available information to the secretary Noël Huybrechts, noel.huybrechts@bbri.be.

The next newsletter will be edited in March 2012.

We wish you all the best for 2012
and hope to see you next year
at the occasion of IS-GI Brussels 2012.

ΕΝΔΙΑΦΕΡΟΝΤΑ ΓΕΩΤΕΧΝΙΚΑ ΝΕΑ

When Nature Destroys in Slow Motion

On May 6, the iris garden alongside Jim and Charity Marlatts' house on a mountain two hours north of Albany was cleaved by a small crack only two inches wide. It was the start of a natural catastrophe, one that is still unfolding at an excruciatingly slow pace.

The home of Jim and Charity Marlatt hangs on the edge of a mudslide in Keene Valley, N.Y.

The Marlatts' treasured glass-and-wood retirement home is now on the scarp — the geologic term for the edge above — of the largest landslide in New York State history. An 82-acre section of earth is slipping downhill and taking trees, rocks and houses with it.

But unlike the landslides that occur in a rush as debris breaks free, usually after a torrential rain or earthquake, this one is occurring incrementally, moving from two inches to two feet per day. "It is like Chinese water torture," Ms. Marlatt said, "drip by drip."

Within weeks, the crack in the garden extended nearly a mile and as the land on the downhill slope began sliding away toward the valley, it created a step that is now a 20-foot vertical drop. Half of the flowers and half of the property beneath the Marlatts' house went with the sliding mass.

Before long, their master bedroom and dining room were hanging over the cliff's edge.

The situation is so precarious that the Marlatts have hired movers to drag their home from the precipice and eventually situate it 100 feet away, a project estimated to cost $150,000. Insurance covers none of it.

Mr. Marlatt said that representatives of New York Central Mutual, their carrier, called it "unfortunate" but that the company did not compensate expenses from damage caused by landslides. They would not send an assessor out to take a look, he said.

Other homeowners — a total of six have so far been affected — have been similarly denied. The American Insurance Association says the risk of landslide is so difficult to predict that routine policies omit damage from such events. Landslides almost always require separate coverage.

Ask and geologists will say that landslides are much more common than people realize. The United States Geological Survey estimates that they occur in every state and each year kill on average 25 people nationally while wreaking $1 billion to $2 billion in damage. Yet they are perhaps the least studied of the natural disasters and difficult to anticipate, said Francis Ashland, a research geologist with the survey.

Slow-moving landslides are also not uncommon, particularly in the West. For example, geologists have been following a very slow slide in North Salt Lake City, Utah, that has been on the move since 1998.

New York State is not generally perceived as landslide territory, but in mountainous areas upstate there are quite a few. The last really large one occurred in Tully Valley, just south of Syracuse, in 1993, during which several homes and dozens of acres of cropland were ruined. Albany and Schenectady have also been the sites of smaller but still costly slides.

The high peaks around Keene and the Finger Lakes Region have been found to be particularly susceptible to the slides because of loose soil deposited on bedrock and formed by glacial lake sediment thousands of years ago, said Andrew Kozlowski, the associate state geologist with the New York State Museum and director of its geological mapping program.

For geologists, figuring out the timing and location of the slides is the real challenge. This has been an active year for slides from Burlington, Vt., to Pittsburgh, said Dr. Ashland, who is studying their underlying cause and rate of occurrence. Western Pennsylvania currently has the highest landslide hazard in the country.

"In general, the likelihood of landslide occurrence increases during wet cycles in which there are more wet years than dry years and periods of successive wet years," he said.

Abnormally heavy snows and spring rains prompted the Keene Valley slide, said Dr. Kozlowski, who is the lead scientist here. To his trained eye, it is clear that there were landslides on this slope before, but hundreds of years ago.

He said this landslide, about a mile wide, was probably set off as groundwater built up and eventually loosened soil as deep as 80 feet. Once surface cracking began, it allowed more water in, and now the whole side of the hill, 60 to 80 feet down, is on the move.

While the actual motion is not detectable to human feet, residents and scientists alike are wary of moving about the slide because occasionally a tree is felled or a boulder is loosened and sent rushing down the slope.

The toe of this landslide, or the bottom, is flowing slowly out on a field in the valley. There, it has already cracked trees and an electric pole and is threatening a farmhouse, about 50 yards away across a dirt road, that Dr. Kozlowski predicts will be damaged within the year.

Steel beams have been placed underneath the Marlatts' home so it can be moved; insurance will not cover the $150,000 cost.

He will not venture a guess as to when the flow might cease. "It could be three months or three years or longer, depending on rainfall," he said.

This house has been condemned due to the mudslide in Keene Valley.

It is theoretically possible to mitigate a slow slide through extensive drainage and by buttressing the toe, but Dr. Ashland said such work was expensive and often did not pass a cost-benefit analysis. He added this would certainly be the case in Keene Valley.

That means homeowners like the Marlatts have little choice but to hope for fine, dry weather.

"We just pray every day for sunshine," Ms. Marlatt said. As she spoke, the sky out of the windows of the guest house darkened with a storm.

(Leslie Kaufman / THE NEW YORK TIMES, June 13, 2011)

ΕΝΔΙΑΦΕΡΟΝΤΑ - ΣΕΙΣΜΟΙ

Βρέθηκε γιατί τα ζώα προβλέπουν τους σεισμούς
Πρωτοποριακή έρευνα Βρετανών και Αμερικανών επιστημόνων

Τα ζώα μπορεί να αντιλαμβάνονται τις χημικές αλλαγές που συμβαίνουν στα επιφανειακά ύδατα λίγο πριν από ένα σεισμό, γεγονός που εξηγεί τη συχνά ανήσυχη συμπεριφορά και την «μαντική» ικανότητά τους, που έχει παρατηρηθεί σε όλη τη Γη από πολύ παλιά. Η νέα αυτή επιστημονική εκτίμηση βασίστηκε στην παρατήρηση μίας ομάδας βατράχων που εγκατέλειψαν την λιμνούλα τους στην περιοχή Λ' Ακουίλα της Ιταλίας, λίγες μέρες πριν λάβει χώρα ο καταστροφικός σεισμός το 2009.

Οι ερευνητές, με επικεφαλής τον γεωφυσικό Φρίντμαν Φρόιντ της NASA και τη βιολόγο Ραχήλ Γκραντ του Ανοικτού Πανεπιστημίου της Βρετανίας, που δημοσίευσαν τη σχετική μελέτη στο διεθνές περιοδικό για θέματα ερευνών περιβάλλοντος και δημόσιας υγείας «International Journal of Environmental Research and Public Health», σύμφωνα με το BBC, πιστεύουν ότι η παρατήρηση της συμπεριφοράς των ζώων θα πρέπει να ενσωματωθεί επίσημα στις μεθόδους πρόβλεψης των σεισμών και συστήνουν τη συνεργασία γεωλόγων και βιολόγων, ώστε να μελετηθεί καλύτερα το θέμα.

Σύμφωνα με τους ερευνητές, τα πετρώματα στον φλοιό της Γης, που υφίστανται ισχυρές τεκτονικές πιέσεις λίγο πριν από ένα σεισμό, απελευθερώνουν φορτισμένα σωματίδια, τα οποία αντιδρούν με τα νερά, προκαλώντας μια σειρά από χημικές μεταβολές. Τα ζώα, που ζουν μέσα στο νερό ή κοντά σε αυτό, έχουν μεγάλη ευαισθησία σε αυτές τις αλλαγές και έτσι είναι σε θέση να τις ανιχνεύσουν ακόμα και μέρες πριν ο σεισμός συμβεί.

Οι βάτραχοι-μετανάστες της Λ' Ακουίλα δεν συνιστούν μεμονωμένη περίπτωση, καθώς στη διάρκεια της παγκόσμιας ιστορίας έχουν αναφερθεί πολλά περιστατικά με διάφορα ζώα (ερπετά, αμφίβια, ψάρια κ.α.) που συμπεριφέρονταν ασυνήθιστα ή έφευγαν μακριά από μία περιοχή όπου σύντομα επρόκειτο να «χτυπηθεί» από σεισμό. Για παράδειγμα, στην Χαϊτσένγκ της Κίνας, το 1975, μέχρι και ένα μήνα πριν συμβεί ένας μεγάλος σεισμός, οι κάτοικοι παρατήρησαν τα φίδια να ξετρυπώνουν μόνα τους από τις φωλιές τους και μάλιστα μέσα στον χειμώνα, δηλαδή στο μέσο της χειμερίας νάρκης τους, αψηφώντας θερμοκρασίες κάτω του μηδενός, παρόλο που κάτι τέτοιο ήταν σχεδόν αυτοκτονία για τα ψυχρόαιμα ερπετά. Το 2009, στο Σαν Ντιέγκο της Καλιφόρνια, οι κάτοικοι είδαν να βγαίνουν στις ακτές δεκάδες καλαμάρια που ζουν σε μεγάλο βάθος (200-600 μέτρων), λίγες ώρες πριν συμβεί ένας μεγάλος σεισμός.

Στην περίπτωση των περίπου 100 βατράχων της Λ' Ακουίλα, η κοινότητά τους εξαφανίστηκε από την περιοχή περίπου τρεις μέρες πριν το σεισμό. Οι ερευνητές έκαναν εργαστηριακά πειράματα και επιβεβαίωσαν ότι οι τεκτονικές δυνάμεις μεταβάλλουν την χημική σύνθεση των γήινων υδάτων. Τα φορτισμένα σωματίδια που απελευθερώνονται από τα συμπιεζόμενα πετρώματα λίγο πριν από ένα σεισμό, μεταδίδονται και στα γειτονικά πετρώματα, ώσπου φθάνουν στην επιφάνεια της Γης και αντιδρούν με τον αέρα της ατμόσφαιρας, μετατρέποντας τα μόρια της σε ιόντα. Αυτά τα αερομεταφερόμενα θετικά φορτισμένα ιόντα, σύμφωνα με προηγούμενες ιατρικές μελέτες, προκαλούν πονοκεφάλους και ναυτία στους ανθρώπους και αυξάνουν στο αίμα των ζώων το επίπεδο της σεροτονίνης, της ορμόνης του στρες, κάνοντάς έτσι πιο ανήσυχα και φοβισμένα.

Παράλληλα, τα ιόντα αυτά αντιδρούν με το νερό, μετατρέποντάς το σε υπεροξείδιο του υδρογόνου. Αυτή η χημική αλυσίδα συμβάντων μπορεί να μετατρέψει τις αβλαβείς οργανικές ύλες μίας λιμνούλας ή της θάλασσας σε τοξικές ουσίες, αναγκάζοντας τα υδρόβια ζώα να σπεύσουν να φύγουν από τη συγκεκριμένη περιοχή. Πρόκειται για ένα πολύπλοκο μηχανισμό που, σύμφωνα με τους επιστήμονες, πρέπει να ερευνηθεί περαιτέρω, ώστε να κατανοηθεί καλύτερα.

Οι ερευνητές ανέφεραν ότι η μελέτη τους φέρνει στο φως για πρώτη φορά με επιστημονικό τρόπο ένα πιθανό γεωλογικό, χημικό και βιολογικό μηχανισμό που, μέσα από διάφορες αλληλεπιδράσεις, τελικά αλλάζει τη συμπεριφορά πολλών ζώων πριν τους σεισμούς, βοηθώντας έτσι στην πρόγνωση αυτών των καταστροφικών φυσικών φαινομένων.

(www.kathimerini.gr με ΑΠΕ-ΜΠΕ, 01.12.2011)

Η συμπεριφορά των φρύνων της Λ' Άκουιλα δεν αποτελεί το πρώτο παράδειγμα περίεργης αντίδρασης ζώων πριν από έναν ισχυρό σεισμό. Μέχρι σήμερα έχουν καταγραφεί και άλλες περιπτώσεις ερπετών, αμφίβιων και ψαριών τα οποία πριν από έναν σεισμό εμφάνιζαν πρωτόγνωρη συμπεριφορά.

Αναλύσεις του Rikitake σε διάφορα ζώα (αγελάδες, σκυλιά, γάτες, ποντίκια, αρουραίους, πουλιά, ψάρια, φίδια, σκουλήκια) δείχνουν ότι υπάρχουν δύο κορυφές στην καμπύλη της ανώμαλης συμπεριφοράς των ζώων, στις 10 και στις 2 ώρες πριν από το σεισμό. Γενικά, η ανώμαλη συμπεριφορά των ζώων παρατηρείται πριν από σεισμούς μεγέθους 5 ή και μεγαλύτερους. Σύμφωνα με τις υπάρχουσες ενδείξεις φαίνεται ότι όσο πιο έντονη είναι η ανώμαλη συμπεριφορά των ζώων, τόσο μεγαλύτερο είναι το μέγεθος του επερχομένου σεισμού. Η πρόβλεψη του μεγάλου σεισμού στο Haicheng της επαρχίας Liaoning της Κίνας, μεγέθους 7.3 Ρίχτερ, στηρίχτηκε και πάνω σε παρατηρήσεις της συμπεριφοράς των ζώων.

Χρήστος Τσατσανίφος (1980) «Έκθεση Διεθνούς Συνεδρίου "ΔΙΕΘΝΕΣ ΣΥΜΠΟΣΙΟ ΓΙΑ ΤΗΝ ΠΡΟΒΛΕΨΗ ΤΩΝ ΣΕΙΣΜΩΝ", Παρίσι, 2-6 Απριλίου 1979», «ΤΕΧΝΙΚΑ ΧΡΟΝΙΚΑ», Ιανουάριος – Φεβρουάριος – Μάρτιος 1980, Πολιτικοί Μηχανικοί – Αρχιτέκτονες – Αγρονόμοι-Τοπογράφοι Μηχανικοί, σελ. 50 – 54.

ΕΝΔΙΑΦΕΡΟΝΤΑ - ΛΟΙΠΑ

Τα ιστορικά χειρόγραφα του Νεύτωνα online στην ψηφιακή βιβλιοθήκη του Cambridge

This is a notebook Newton acquired while he was an undergraduate at Trinity College and used from about 1661 to 1665 (http://cudl.lib.cam.ac.uk/view/MS-ADD-03996). It includes many notes from his studies and, increasingly, his own explorations into mathematics, physics and metaphysics. It was judged 'Not fit to be printed' by Newton's executor and was presented to the Library by the fifth Earl of Portsmouth in 1872.

This notebook contains many blank pages (all shown) and has been used by Newton from both ends. Our presentation displays the notebook in a sensible reading order. It shows the 'front' cover and the 30 folios that follow and then turns the notebook upside down showing the other cover, and the pages that follow it. Full transcriptions are available for folios 88r-135r, a famous section of the manuscript where Newton organises his notetaking according to 'Questiones quaedam Philosophiae' (certain philosophical questions). The notebook was photographed while it was disbound in 2011.

Cambridge University Library, Cambridge, UK

Το πανεπιστήμιο Κέμπριτζ επιτρέπει πλέον για πρώτη φορά την διαδικτυακή πρόσβαση οποιουδήποτε ενδιαφερόμενου στα ψηφιοποιημένα χειρόγραφα και πρωτότυπα τυπωμένα έργα του μεγάλου επιστήμονα. Μεταξύ αυτών βρίσκεται η πρωτότυπη τυπωμένη έκδοση του αριστουργήματός του «Principia Mathematica» (Μαθηματικές Αρχές της Φυσικής Φιλοσοφίας), μαζί με τις εμβόλιμες σχετικές χειρόγραφες σημειώσεις και απαντητικά σχόλια στους επικριτές του, που ο ίδιος είχε κάνει πάνω στο δικό του αντίτυπο.

Μέχρι στιγμής, σύμφωνα με τη βρετανική «Γκάρντιαν», περισσότερες από 4.000 σελίδες, δηλαδή περίπου το ένα πέμπτο του αρχείου του Νεύτωνα, που διατηρεί το φημισμένο πανεπιστήμιο, έχουν ψηφιοποιηθεί και είναι προσβάσιμα online στο πλαίσιο ενός προγράμματος, το οποίο θα δώσει στο ευρύ κοινό πρόσβαση στο έργο και άλλων «κολοσσών» της επιστήμης, όπως ο Δαρβίνος.

Όπως δήλωσε ο υπεύθυνος για την ψηφιοποίηση στη βιβλιοθήκη του Κέμπριτζ, Γκραντ Γιανγκ, τα χειρόγραφα του Νεύτωνα αποκαλύπτουν τον τρόπο που σκεπτόταν και σταδιακά προχωρούσε στις σημαντικές ανακαλύψεις του, που σφράγισαν τη σύγχρονη επιστήμη.

Για ρίξτε όμως και μια ματιά και στο σημειωματάριό του. Αναγνωρίζετε τη γλώσσα που χρησιμοποιούσε; (http://cudl.lib.cam.ac.uk/view/MS-ADD-03996/9).

Μπορείτε να ψάξετε σελίδα - σελίδα τις 285 σελίδες αυτών των χειρογράφων του Ισαάκ Νεύτωνα, να μεγεθύνετε κάθε σελίδα όπως θέλετε (υπάρχουν σχετικά κουμπιά ελέγχου) και να διαβάσετε την Ελληνική γλώσσα που χρησιμοποίησε παράλληλα με την Αγγλική.

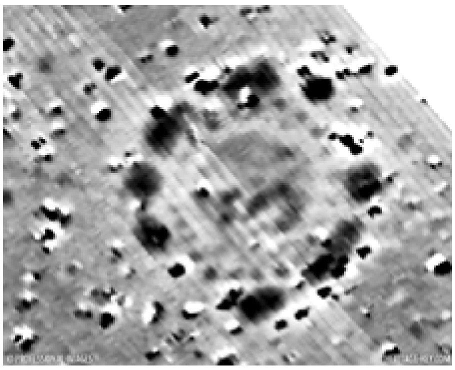
Γιορτή του νεολιθικού ηλιοστάσιου

Ακόμα αρχαιότερος «ναός του Ήλιου» κρυβόταν στο υπέδαφος του Στόουνχεντζ

Ακόμα και σήμερα, εκατοντάδες άνθρωποι γιορτάζουν στο Στόουνχενζ το θερινό ηλιοστάσιο

Χρησιμοποιώντας εξελιγμένες τεχνικές απεικόνισης του υπεδάφους, διεθνής ομάδα αρχαιολόγων εντόπισε κοντά στο κεντρικό μνημείο του Στόουνχεντζ ενδείξεις για ένα ακόμα παλαιότερο χώρο λατρείας, στον οποίο φαίνεται ότι εορταζόταν το θερινό ηλιοστάσιο.

Οι ερευνητές ανακάλυψαν τα θαμμένα ίχνη δύο λάκκων, οι οποίοι ευθυγραμμίζονταν με τον Ήλιο την ώρα της ανατολής και της δύσης, όταν κανείς τις κοιτάζει από τη θέση ενός ογκόλιθου κοντά στην σημερινή κεντρική είσοδο του αρχαιολογικού χώρου. Αυτό συνέβαινε μόνο κατά το θερινό ηλιοστάσιο, τη μεγαλύτερη μέρα του χρόνου.



Τα νέα ευρήματα δεν έχουν χρονολογηθεί με ακρίβεια, είναι όμως προφανές ότι είναι παλαιότερα από το διάσημο νεολιθικό μνημείο του Στόουνχεντζ, ηλικίας περίπου 5.000 ετών. Η περιοχή δεν έχει ανασκαφεί, ωστόσο οι λάκκοι είναι ορατοί σε εικόνες από μαγνητόμετρα (*παράδειγμα στην ένθετη αριστερά*) και άλλα όργανα απεικόνισης του υπεδάφους.

Όπως ανακοίνωσαν οι αρχαιολόγοι του Πανεπιστημίου του Μπέρμιγχαμ, οι οποίοι συνεργάζονται με συναδέλφους τους στη Βιέννη, οι δύο λάκκοι πιστεύεται ότι ήταν τα σημεία στα οποία ξεκινούσε και τερμάτιζε μια πομπή που ακολουθούσε την κίνηση του Ήλιου στη διάρκεια της ημέρας.

«Τα συναρπαστικά αυτά ευρήματα υποδεικνύουν ότι, παρόλο που το Στόουνχεντζ έγινε τελικά το σημαντικότερο μνημείο του τοπίου, δεν αποκλείεται να μην ήταν πάντα το μόνο, ή το σημαντικότερο, τελετουργικό σημείο της περιοχής και ενδέχεται να έγινε σημαντικό ως ιερός τόπος παρά πολύ αργότερα» αναφέρει ο καθηγητής Βινς Γκάφνι, επικεφαλής της μελέτης από το Κέντρο Οπτικής και Χωρικής Τεχνολογίας IBM στο Πανεπιστήμιο του Μπέρμιγχαμ.

Οι λάκκοι, μέσα στους οποίους εκτιμάται ότι μπορεί να υπήρχαν ξύλινοι στύλοι, ογκόλιθοι ή φωτιές που σημάδευαν τα σημεία, βρέθηκε εντός του «Νεολιθικού Cursus», μιας γνωστής διαδρομής που ορίζεται από δύο παράλληλα χαντάκια.

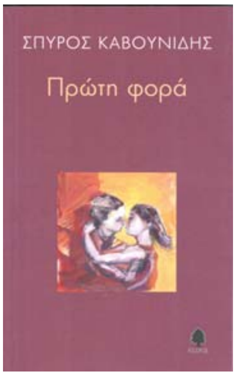
«Είναι πιθανό οι πομπές εντός του Cursus να κινούνταν από τον ανατολικό λάκκο το ξημέρωμα, συνεχίζοντας ανατολικά κατά μήκος του Cursus και, ακολουθώντας την πορεία του Ήλιου, κινούνταν πίσω προς τη δύση, φτάνοντας στον δυτικό λάκκο το ηλιοβασίλεμα, γιορτάζονταν τη μεγαλύτερη μέρα του χρόνου» εικάζει ο Δρ Γκάφνι.

Η ομάδα του καθηγητή ερευνά το υπέδαφος της περιοχής από το 2010, στο πλαίσιο του ερευνητικού Προγράμματος Κρυμμένων Τοπίων του Στόουνχεντζ.

(Newsroom ΔΟΛ, 28 Νοε. 2011)

ΝΕΕΣ ΕΚΔΟΣΕΙΣ ΣΤΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΠΙΣΤΗΜΕΣ

Βιβλίο όχι γεωτεχνικής μηχανικής αλλά από γεωτεχνικό μηχανικό…



Την Δευτέρα, 12 Δεκεμβρίου 2011, παρουσιάστηκε στο βιβλιοπωλείο ΙΑΝΟΣ το βιβλίο του πρώην Προέδρου της ΕΕΕΕΓΜ και μέλους της Εκτελεστικής Επιτροπής Σπύρου Καβουνίδη «Πρώτη φορά». Η παρουσίαση του βιβλίου έγινε από την δημοσιογράφο της εφημερίδας «Η ΚΑΘΗΜΕΡΙΝΗ» Μαρία Καστουνάκη και τον ποιητή Τίτο Πατρίκιο. Παραθέτουμε, στη συνέχεια, το κείμενο της παρουσίασης της Μαρίας Κατσουνάκη.

Θα ξεκινήσω παραθέτοντας ένα μικρό απόσπασμα από τον πρόλογο του Σπύρου Καβουνίδη στο βιβλίο του «Πρώτη φορά»: «Πρώτες φορές υπάρχουν πολλές, πάμπολες στη ζωή. Αυτές που καταγράφονται εδώ δεν είναι αναγκαστικά οι σημαντικότερες. Όπως θα 'ταν, για παράδειγμα, η πρώτη μερα στο σχολείο, ο πρώτος έρωτας, η πρώτη μέρα στη δουλειά, η πρώτη μέρα του παιδιού σου. Αυτό το βιβλίο αποτελείται από δεκαεννέα διηγήματα, δεκαεννέα στιγμιότυπα. Αρκετά είναι αυτοβιογραφικά, ορισμένα είναι εμπνευσμένα από διηγήσεις άλλων, έστω και αν όλα είναι γραμμένα σε πρώτο πρόσωπο, σε πολλά έχει εισχωρήσει και η φαντασία».

Συνεχίζω με κάτι με κάτι που είπε ο Τίτος Πατρίκιος σε συνέντευξή του στο Βήμα πριν από λίγες ημέρες με αφορμή την έκδοση της νέας του ποιητικής συλλογής «Συγκατοίκηση με το παρόν»: «Μας αρέσει - δεν μας αρέσει, είμαστε αναγκασμένοι να συγκατοικούμε με το παρόν, γιατί μόνον έτσι μπορούμε να δούμε πού βρισκόμαστε και ενδεχομένως να χαράξουμε, έστω αμυδρά, έναν δρόμο για το πού θα πάμε. Έχω την αίσθηση ότι η ανάκληση του παρελθόντος γίνεται πια όχι για να το επισκοπήσουμε κριτικά αλλά για να το νοσταλγήσουμε και κατά έναν φαντασιακό τρόπο να επιστρέψουμε στο παρελθόν».

Αναρωτήθηκα αν αυτό χαρακτηρίζει τα 19 αφηγήματα του Σπύρου Καβουνίδη που ξεκινούν το 1944 και έχουν ανοιχτή ημερομηνία λήξης, μιας και το τελευταίο επιγράφεται «Η πρώτη μου κηδεία»! Σε αυτό το τελευταίο, 19^{ο} αφήγημα, ο Σπύρος Καβουνίδης μας κάνει συμμέτοχους και συνένοχους στην κηδεία του. Προσφιλές παιχνίδι της φαντασίας και του εγώ, που πουθενά δεν βολεύεται και διεκδικεί πάντα την καλύτερη θέση για ένα πανοραμικό φινάλε. Η εικονική κηδείας μας ως υπερπαραγωγή, ανάλογα με τις απαιτήσεις του καθενός μας, δηλώνει την εικόνα που έχουμε για τον εαυτό μας, κατανέμει ευθύνες και ρόλους στους άλλους. Είναι η στιγμή της υπέρτατης (αυτό) ικανοποίησης. Το ομολογεί και ο ίδιος ο συγγραφέας: «Σας το συνιστώ αυτό το όνειρο φαντασία που ανανεώνεται. Προσφέρει εσωτερική διέξοδο στο ναρκισσισμό, για να μην ξεφτιλιζόμαστε κιόλας, και αφήνει πάντα ανοιχτές προοπτικές για το επόμενο. Η ζωή χρειάζεται τις φανταστικές κηδείες».

Κι εδώ, κατά τη γνώμη μου, μπαίνει η πιο ειλικρινής και αποτελεσματική πινελιά: Η κάθε πρώτη φορά είναι ταυτόχρονα και η τελευταία. Οσες επαναλήψεις και βελτιώσεις κι αν επιδέχεται, όσο εξωραϊστικό ή αποδομητικό κι αν είναι το παιχνίδι της μνήμης, η κάθε γέννηση – καταγραφή της στιγμής, συνυπάρχει με το θάνατο της. Καλωσόρισμα και αποχαιρετισμός.

Στο ανθολόγιο αυτό από «πρώτες φορές», καθόλου αναμενόμενες, όπως διευκρινίζεται και στον πρόλογο, ο Σπύρος Καβουνίδης κάνει, αυτό που θα λέγαμε με κινηματογραφικούς όρους, ένα πρωτότυπο ντεκουπάζ. Ντεκουπάζ είναι η κατάτμηση του σεναρίου σε πλάνα, ο τεμαχισμός με σχετική ακρίβεια της δράσης του αφηγήματος σε πλάνα, σκηνές και σεκάνς πριν το γύρισμα**.**

Παρακολουθούμε λοιπόν την έναρξη της «ταινίας», Δεκέμβρη του '44, με τη γέννηση του Σπύρου στην Αθήνα, με τη συμβολή και προστασία των Μεγάλων Δυνάμεων (!), και συνεχίζουμε με μικρές ή μεγαλύτερες δρασκελιές σε διαδοχικές γενέθλιες συναντήσεις: Το φιλί, το ροκ εντ ρολ, το μεθύσι, το κουστούμι. Όλα πρώτα. Καθόλου όμως μυθοποιημένα. Με ειλικρίνεια, χιούμορ, αυτοσαρκασμό και ένα ισοζύγιο συναισθημάτων: ίσες δόσεις μελαγχολίας, πραγματικότητας και λογοτεχνικής μεταγραφής της, σύνθεσης και αποδόμησης. Φταίει η μνήμη γι αυτό, η ηλικία, ή το γούστο που σφραγίζει κάθε εποχή; Εχουμε και λέμε «Φιλί», 1953: ύστερα από χρόνια το υπόξανθο εντυπωσιακό μαλλί της Δώρας μπορεί να ήταν και ξεπλυμένο, τα υπέροχα μπλε μάτια ξεθωριασμένα! «Να μη σας πω ότι ίσως αλληθώριζε κιόλας»…

Οσο για την πρώτη «Χυλόπιτα» ήταν το 1960 σε ηλικία 15 χρονών. Εκτοτε, μας λέει ο Σπύρος Καβουνίδης, ακολούθησαν… και άλλες! Το 1963, η χρονιά της Λίνας, συνδέεται με τον ρομαντικό έρωτα αλλά και το άδοξο τέλος της απόπειρας του συγγραφέα να γράψει ποίηση.

Η διαδρομή στην οποία μας προσκαλεί σηματοδοτείται από γυναίκες. Η αμεσότητα, χαλαρότητα και καθαρότητα της περιγραφής κάτι μας λέει για την καθοριστική επίδραση στη ζωή του Σπύρου Καβουνίδη της Γενικής Γραμματείας Ισότητας των Φύλων!!!

Δεν χρειάζεται να γνωρίζει κανείς προσωπικά το Σπύρο για να καταλάβει ως αναγνώστης ότι η θηλυκή πλευρά, η οποία και χαρίζει στον άντρα γοητεία, πολυμορφία και ευθραυστότητα, αποτελεί στοιχείο του χαρακτήρα του, του τρόπου που κοιτάζει (όχι βλέπει, κοιτάζει) τους ανθρώπους και τη ζωή.

Η 16^{η} ιστορία έχει τίτλο η «Πρώτη της φορά» (1972). Πρωταγωνιστεί η Μελίσσα και ζει την πρώτη της ερωτική εμπειρία. Δεν θα επιμείνω στον τρόπο που βιώνεται και περιγράφεται από τον γράφοντα γιατί θα προδώσω την ανατροπή. Άλλο θέλω να πω. Πως καμιά φορά οι «πρώτες φορές» αλληλοδιαπλέκονται και αλληλοεπηρεάζονται. Πως η πρώτη φορά του άλλου, καρποφορεί και στη δική μας πραγματικότητα, με τρόπο ευχάριστο ή δυσάρεστο, δεν έχει σημασία.

Ο Σπύρος Καβουνίδης δεν ωραιοποιεί, δεν αποφεύγει την αυτοκριτική (για αυτολογοκρισία δεν είμαι σε θέση να μιλήσω!), γράφει ελεύθερα. Εν ολίγοις εκτίθεται. Και η έκθεση έχει ρίζες και έννοιες υπαρξιακές. Διότι οι άνθρωποι, απλοί ή εκλεπτυσμένοι, ευφυείς ή ηλίθιοι, έρχονται συνεχώς αντιμέτωποι στη ζωή τους με το ωραίο, με το άσχημο, με το κιτς ή με το ευτελές, με το υπέροχο, με το κωμικό, με το τραγικό, με το λυρικό, με το δραματικό, με τη δράση, με τις περιπέτειες, με την κάθαρση, όπως λέει και ο Κούντερα. Ολες αυτές οι έννοιες είναι δρόμοι που οδηγούν σε διάφορες πλευρές της ύπαρξης, απρόσιτες με οποιοδήποτε άλλο μέσο.

Ετσι λοιπόν κι εδώ ο Σπύρος διάλεξε, άραγε, τον δικό του τρόπο για να επιστρέψει στο παρελθόν και να ξαναδιαβάσει το παρόν; Και ναι και όχι. Υπάρχει νοσταλγία σε όλες αυτές τις πρώτες φορές; Θα έλεγα αμεριμνησία, ανεμελιά. Ενας τρόπος για να αποφύγω την πολυφορεμένη λέξη «αθωότητα». Ακόμη και όταν μιλάει για στιγμές πιο δύσκολες, λίγο ζόρικες, δεν δραματοποιεί. Αναζητάει την αλήθεια εκ των υστέρων, ερμηνεύει με τα εργαλεία που απέκτησε στο χρόνο. Διαβάζω από την «Πρώτη νύχτα», Μάρτιος 1974: «Πώς θα με ανακρίνουν και για ποιο πράγμα; Θα με χτυπήσουν; Κα πόσο θα κρατούσα; Θα ομολογούσα άραγε; Αλλά τι να ομολογήσω; Δεν είχα προλάβει δα να κάνω και τίποτα σοβαρό. Και το σκουληκάκι στο πάτωμα από πού βρέθηκε, από πού τρύπωσε; Και ο γλόμπος στο ταβάνι πόσα Watt ήταν; Κι αν έσβηνε θα τον αντικαθιστούσαν άραγε ή θα 'μενα στο σκοτάδι; Μήπως όμως προτιμούσα το σκοτάδι; Σκέφτηκα ότι μισούσα τον υπαξιωματικό, αυτόν που είχα νιώσει ότι με μισεί. Τον μισούσα όμως ή τον φοβόμουν; Η τον μισούσα επειδή τον φοβόμουν;».

Η «Πρώτη φορά» είναι άραγε και μια εξομολόγηση; Μαθαίνουμε κάτι περισσότερο προσωπικό για τον συγγραφέα, για τις συνήθειες, την προσωπικότητά του, τον μηχανισμό της σκέψης και τα συναισθήματά του; Διαβάζοντας δηλαδή τα 19 αφηγήματα τρυπώνουμε στη ζωή του Σπύρου Καβουνίδη από μια κερκόπορτα που ο ίδιος ανοίγει για να μας κάνει κοινωνούς; Είναι αξιοθαύμαστο πάντως το γεγονός ότι αυτός ο αθεράπευτος ροκάς μπόρεσε να περιορίσει το πάθος του σε δυο μόνο από τα 19 διηγήματα! 1957 και μια λάθος φιγούρα στο ροκ εντ ρολ αποβαίνει μοιραία!... 1969 και η ανεπανάληπτη εμπειρία της ροκ συναυλίας στο Αλταμοντ στη Βόρεια Καλιφόρνια.

Διασχίζουμε μαζί του τρεις δεκαετίες '50, '60 και '70– καθοριστικές για τον πλανήτη, τη μεγάλη κλίμακα, αλλά και τη μικρή, μικρότατη της ζωής του ενός, του καθενός. Για το Σπύρο, τρεις δεκαετίες γεμάτες αισθήσεις, εικόνες, συναντήσεις. Ανάμεσα σε χνώτα, ανθρώπους, μυρωδιές. Σαν λήψεις από ερασιτεχνική κάμερα. Ο σκηνοθέτης επιθυμεί κυρίως να μας διηγηθεί ιστορίες, που μπορεί να μας ενώνουν ή να μας χωρίζουν, λίγη έχει σημασία. Εκείνο που μετράει είναι ότι μας καλεί αναδρομικά σε μια γιορτή. Όχι γιατί ό,τι διηγείται έχει εορταστικό περιτύλιγμα. Κάθε άλλο. Αλλά γιατί μας συμφιλιώνει με αυτό που ζήσαμε και δεν γνωρίζαμε ότι και άλλοι έχουν ζήσει ή με αυτό που θα θέλαμε να έχουμε ζήσει και κάποιοι ήταν εκεί και το μοιράζονται μαζί μας.

Σκέφτομαι συχνά: είμαστε πολλών ανθρώπων άνθρωπος.

Ο Σπύρος Καβουνίδης το συμμερίζεται.

Λίγο πριν τελειώσω θα ήθελα να αναφερθώ και πάλι στον Τιτο Πατρίκιο. Λέει: «Κι αν γυρίζει ο χρόνος πίσω / στην αρχή/ όπως γυρίζει πίσω μια βιντεοκασέτα το ίδιο αδέξια, με την ίδια ταραχή τα ίδια θα κάναμε, ας τα και γάμησέ τα»…..

Τελειώνω με μιαν ευχή. Σήμερα ο Σπύρος γιορτάζει. Θάθελα η δική μου ευχή να είναι μια προτροπή. Αυτήν που απηύθυνε ο Στιβ Τζομπς στους αποφοίτους του Στάνφορντ το 2005. Κακόπαθε στην ελληνική της μετάφραση, αλλά εγώ θα επιμείνω γνωρίζοντας ότι απευθύνομαι σε δεινούς γνώστες της αγγλικής. «**Stay hungry. Stay foolish».** Κράτα τη δίψα σου, κράτα την τρέλα σου Σπύρο. Μείνε νέος.

Μαρία Κατσουνάκη

ΔΙΑΘΕΣΗ ΕΥΡΩΚΩΔΙΚΩΝ

Κατόπιν παρέμβασης του Συλλόγου Πολιτικών Μηχανικών Ελλάδας, ο ΕΛΟΤ αποφάσισε να διαθέσει τα κείμενα των Ευρωκωδίκων στα μέλη του ΣΠΜΕ σε **σημαντικά χαμηλότερη τιμή** από αυτή που είχε ανακοινωθεί έως σήμερα. Η προσφορά του ΕΛΟΤ έχει ως ακολούθως:

Βασικό Πακέτο (περιλαμβάνει τα κείμενα για τους ευρωκώδικες : ΕΚ0, ΕΚ1, ΕΚ2 και ΕΚ8) μαζί με τα αντίστοιχα προσαρτήματα. Η τιμή διάθεσης των παραπάνω είναι :

- 180 € για τα φυσικά πρόσωπα. Η έκδοση αυτή προσφέρεται μόνο Read Only, χωρίς δυνατότητα εκτύπωσης.
- 360 € για τα νομικά πρόσωπα. Η έκδοση αυτή προσφέρεται Read Only, χωρίς εκτύπωση, με δυνατότητα εμφάνισης στο εσωτερικό δίκτυο του αγοραστή (μέχρι 5 PC).

Πλήρες Πακέτο (περιλαμβάνει τα κείμενα για τους ευρωκώδικες : ΕΚ0 έως ΕΚ9) μαζί με τα αντίστοιχα προσαρτήματα. Η τιμή διάθεσης των παραπάνω είναι :

- 360€ για τα φυσικά πρόσωπα. Η έκδοση αυτή προσφέρεται μόνο Read Only, χωρίς δυνατότητα εκτύπωσης.
- 540€ για τα νομικά πρόσωπα. Η έκδοση αυτή προσφέρεται Read Only, χωρίς εκτύπωση, με δυνατότητα εμφάνισης στο εσωτερικό δίκτυο του αγοραστή (μέχρι 5 PC).

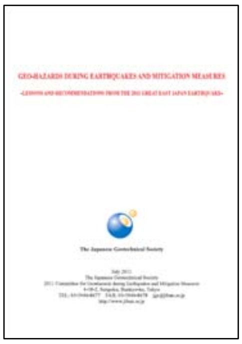
Ετήσια συνδρομή για την ανανέωση των κειμένων 20 € / έτος

Προϋπόθεση για να ισχύσει η παρούσα προσφορά είναι να **προεγγραφούν 5,000 μέλη του ΣΠΜΕ και τεχνικών εταιρειών**.

Η προσφορά θα ισχύσει για συγκεκριμένο χρονικό διάστημα.

Πληροφορίες και λεπτομέρειες για τον τρόπο προεγγραφής και αγορά των ευρωκωδίκων στην ιστοσελίδα του ΣΠΜΕ (www.spme.gr/index.php?&sec=&cid=451).

ΣΥΛΛΟΓΟΣ ΠΟΛΙΤΙΚΩΝ ΜΗΧΑΝΙΚΩΝ ΕΛΛΑΔΑΣ, Ιπποκράτους 9, ΑΘΗΝΑ, Τ.Κ. 106 79, τηλ. 210.9238170, τοτ. 210. 9235959, ηλ.δι. spme@tee.gr.



Geo-hazards During Earthquakes and Mitigation Measures

Publication of the Japanese Geotechnical Society

The Japanese Geotechnical Society has published "GEO-HAZARDS DURING EARTHQUAKES AND MITIGATION MEASURES - LESSONS AND RECOMMENDATIONS FROM THE 2011 GREAT EAST JAPAN EARTHQUAKE".

This document is based on experiences with, and lessons from, the disasters that resulted from the 2011 Great East Japan Earthquake, 11 March 2011. These documents show several fields where geosynthetics engineering can be applied for restoration, reconstruction and new construction.

The full version will be available to be downloaded from http://www.jiban.or.jp/file/e/pub/lessonandrecommendation2011.pdf.

(November 2011)

ΗΛΕΚΤΡΟΝΙΚΑ ΠΕΡΙΟΔΙΚΑ

International Society for Rock Mechanics

ISRM newsletter

No. 16 - December 2011

http://www.isrm.net/adm/newsletter/ver_html.php?id_newsletter=67&ver=1

Κυκλοφόρησε το Τεύχος 16 / Δεκέμβιος 2011 του Newsletter της International Society for Rock Mechanics. Περιεχόμενα:

- President's 2012 New Year Address
- New Board of the ISRM elected for 2011-2015
- The 50th anniversary of the ISRM
- The first group of Fellows of the ISRM was inducted in Beijing
- ISRM International Symposium Eurock 2012, Stockholm, 28-30 May
- 2nd SASORE, Costa Rica, 7 to 9 August 2012
- ARMS 7, Seoul, 15 to 19 October 2012
- New ISRM Membership Management System

- Access to the ISRM Digital Library with OnePetro ISRM sponsored meetings
- The 12th ISRM Congress in Beijing was a great success
- 2nd ISRM International Young Scholars' Symposium on Rock Mechanics
- Jinping II - The world's largest tunnel group is broken through
- International Journal of the Japanese Committee for Rock Mechanics
- CRC Press / Balkema - Special Online Offer for ISRM Members

geoengineer

www.geoengineer.org

Κυκλοφόρησαν το Τεύχος #83 του **Newsletter του Geoengineer.org** (Δεκέμβριος 2011) με πολλές χρήσιμες πληροφορίες για όλα τα θέματα της γεωτεχνικής μηχανικής. Υπενθυμίζεται ότι το Newsletter εκδίδεται από τον συνάδελφο και μέλος της ΕΕΕΕΓΜ Δημήτρη Ζέκκο (secretariat@geoengineer.org).

ASSOCIATION INTERNATIONALE DES TUNNELS ET DE L'ESPACE SOUTERRAIN AITES | ITA INTERNATIONAL TUNNELLING AND UNDERGROUND SPACE ASSOCIATION

INTERNATIONAL TUNNELLING AND UNDERGROUND SPACE ASSOCIATION

ita@news n°42

http://ita-aites.org/index.php?id=886&no_cache=1

Κυκλοφόρησε το Τεύχος No. 42 – Δεκέμβριος 2011 των ita@news της International Tunnelling Association.

ITACET Foundation

Foundation for Education and Training on Tunnelling and Underground Space Use

http://www.itacet.org/Newsletter/05_2011/newsletter_5_2011.php

Κυκλοφόρησε το Τεύχος No. 10 (Δεκέμβριος 2011) του ITACET Foundation.

IGS NEWS

NEWSLETTER OF THE INTERNATIONAL GEOSYNTHETICS SOCIETY

Dedicated to the scientific and engineering development of geotextiles, geomembranes, related products, and associated technologies

VOLUME 27, NO. 3 2011

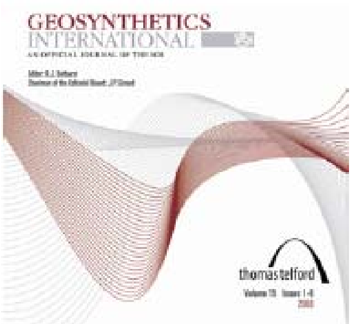
http://library.constantcontact.com/download/get/file/1103777414955-100/2011-11-igs-news-a8+Dec+19.pdf

Κυκλοφόρησε το Τεύχος 3, Volume 27 των IGS News. Μεταξύ των θεμάτων περιλαμβάνονται:

- Νέα για τις σχέσεις της IGS με τις Αδελφές Διεθνείς Επιστημονικές Ενώσεις.
- Πρόσκληση υποβολής υποψηφιοτήτων για το Συμβούλιο της IGS Council για την περίοδο 2012 έως 2016.
  Η προθεσμία για την υποβολή συμμετοχών στον IGS "2nd Photo Contest"

Περίληψη των δραστηριοτήτων των Εθνικών Ενώσεων της IGS στο 2010

- Η συμμετοχή της IGS στην Federation of the International Geo-Engineering Societies
- Το Memorandum of Understanding μεταξύ της IGS και της ICID (International Congress on Irrigation and Drainage)
- Πληροφορίες για τα Συνέδρια του 2012



Geosynthetics International

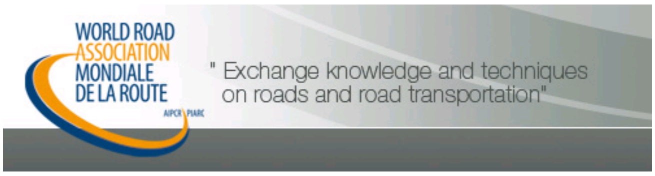
www.thomastelford.com/journals

Κυκλοφόρησε το τεύχος αρ. 4 του 18ου τόμου (Αυγούστου 2011) του περιοδικού Geosynthetics International. Πρόσβαση μέσω της ιστοσελίδας http://www.icevirtuallibrary.com/content/issue/gein/18/4.

Geotextiles & Geomembranes

www.geosyntheticssociety.org/journals.htm

Κυκλοφόρησαν τα τεύχη αρ. 5 και 6 του 29ου τόμου (Οκτωβρίου και Δεκεμβρίου 2011) καθλως και το τεύχος 1 του 30ου τόμου (Φεβρουάσριος 2012) του περιοδικού Geotextiles & Geomembranes. Πρόσβαση μέσω της ιστοσελίδας www.sciencedirect.com/science/journal/02661144.



www.piarc.org

http://www.piarc.org/ressources/documents/11541,Newsletter26-EN-September2011.pdf

http://www.piarc.org/ressources/documents/11669,Newsletter27-EN-December2011.pdf

Κυκλοφόρησε το Τεύχος No. 42 (Δεκέμβριος 2011) του **Newslet-ter της World Road Association** (PIARC) και τα Τεύχη No. 26 (Σεπτέμβριος 2011) και No. 27 (Δεκέμβριος 2011) του Newsletter των PIARC National Committees.

ΕΕΕΕΓΜ
Τομέας Γεωτεχνικής
ΣΧΟΛΗ ΠΟΛΙΤΙΚΩΝ ΜΗΧΑΝΙΚΩΝ
ΕΘΝΙΚΟΥ ΜΕΤΣΟΒΙΟΥ ΠΟΛΥΤΕΧΝΕΙΟΥ
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«ΤΑ ΝΕΑ ΤΗΣ ΕΕΕΕΓΜ» «αναρτώνται» και στην ιστοσελίδα www.hssmge.gr